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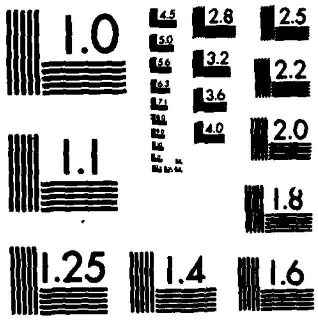
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DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

AUGUST, 1979

UNCLASSIFIED

SECURITY CLASSIFICATION OF THIS PAGE (When Data Entered)

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Cover program reads: Phase I Inspection Report, National Dam Inspection Program; however, the official title of the program is: National Program for Inspection of Non-Federal Dams; use cover date for date of report.

19. KEY WORDS (Continue on reverse side if necessary and identify by block number)

DAMS, INSPECTION, DAM SAFETY,

Connecticut Coastal Basin
Woodbridge, Connecticut

20. ABSTRACT (Continue on reverse side if necessary and identify by block number)

The earth embankment dam is approximately 960± feet long and rises approximately 48 feet above the downstream bed of the West River. Based upon the visual inspection at the site and past performance, the dam appears to be in good condition. Based upon the size (Intermediate) and hazard classification (High) of the dam in accordance with Corps of Engineers Guidelines, the test flood will be equivalent to the Probable Maximu Flood (PMF).

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DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02154

REPLY TO ATTENTION OF:
NEDED

NOV 28 1979

Honorable Ella T. Grasso
Governor of the State of Connecticut
State Capitol
Hartford, Connecticut 06115

Dear Governor Grasso:

Inclosed is a copy of the Lake Dawson Dam Phase I Inspection Report, which was prepared under the National Program for Inspection of Non-Federal Dams. This report is presented for your use and is based upon a visual inspection, a review of the past performance and a brief hydrological study of the dam. A brief assessment is included at the beginning of the report. I have approved the report and support the findings and recommendations described in Section 7 and ask that you keep me informed of the actions taken to implement them. This follow-up action is a vitally important part of this program.

A copy of this report has been forwarded to the Department of Environmental Protection, the cooperating agency for the State of Connecticut. In addition, a copy of the report has also been furnished the owner, New Haven Water Company, New Haven, Connecticut 06511.

Copies of this report will be made available to the public, upon request, by this office under the Freedom of Information Act. In the case of this report the release date will be thirty days from the date of this letter.

I wish to take this opportunity to thank you and the Department of Environmental Protection for your cooperation in carrying out this program.

Sincerely,

MAX B. SCHEIDER
Colonel, Corps of Engineers
Division Engineer

Incl
As stated

CONNECTICUT COASTAL BASIN

WOODBRIDGE, CONNECTICUT

LAKE DAWSON DAM

CT 00319

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY

NEW ENGLAND DIVISION, CORPS OF ENGINEERS

WALTHAM, MASS. 02154

AUGUST, 1979

BRIEF ASSESSMENT

PHASE I INSPECTION REPORT

NATIONAL PROGRAM OF INSPECTION OF DAMS

Name of Dam	LAKE DAWSON DAM
Inventory Number:	CT-00319
State Located:	CONNECTICUT
County Located:	NEW HAVEN
Town Located:	WOODBRIDGE
Stream:	WEST RIVER
Owner:	NEW HAVEN WATER COMPANY
Date of Inspection:	MAY 1, 1979
Inspection Team:	PETER M. HEYNEN, P.E.
	CALVIN GOLDSMITH
	MIRON PETROVSKY
	GEORGE STEPHENS

The earth embankment dam is approximately 960$\pm$ feet long and rises approximately 48 feet above the downstream bed of the West River. A concrete corewall apparently runs the length of the dam. The concrete spillway at the left end of the dam is a 110 foot long broad crested weir with a vertical-sided concrete channel leading to a steep-sided channel cut into natural ground. An upstream gatehouse near the right end of the dam houses the regulating outlets which consist of 2-36 inch cast iron low level outlets, a 36 inch supply main to the treatment plant immediately downstream of the dam, and an 8 inch well drain.

Based upon the visual inspection at the site and past performance, the dam appears to be in good condition. No evidence of instability was observed in the embankment, spillway, spillway channel, or other appurtenances.

Based upon the size (Intermediate) and hazard classification (High) of the dam in accordance with Corps of Engineers Guidelines, the test flood will be equivalent to the Probable Maximum Flood (PMF). Peak inflow to the reservoir is 20,100 cfs; peak outflow is 19,000 cfs with the dam overtopped 1.7 feet. Based upon our hydraulic computations, the spillway capacity is 9900 cfs, which is equivalent to 52 percent of the routed test flood outflow.

It is recommended that the owner initiate further studies to perform a more refined hydraulic/hydrologic study by a qualified engineer to determine more accurately the spillway capacity and overtopping potential. Recommendations should be made by the engineer and implemented by the owner to increase the project discharge capacity if called for by the refined hydraulic/hydrologic study.

It is further recommended that the owner initiate an investigation by a registered engineer qualified in dam design, hydraulics, and inspection to 1) evaluate and make recommendations to monitor, control, and/or eliminate the seepage emanating from the downstream area of the dam, 2) assess the amount and seriousness of uplift pressure exerted on the concrete spillway channel slab, and 3) develop a program of repairs for leaking expansion joints and cracks in the concrete spillway and spillway slab.

The above recommendations, and any needed remedial measures are further discussed in Section 7, and should be instituted by the owners within two years of their receipt of this report.

Peter M Heynen

Peter M. Heynen, P.E.
Project Manager
Cahn Engineers, Inc.

Edgar B. Vinal Jr.

Edgar B. Vinal, Jr., P.E.
Senior Vice President
Cahn Engineers, Inc.

This Phase I Inspection Report on Lake Dawson Dam has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and is hereby submitted for approval.

Joseph W. Finegan

JOSEPH W. FINEGAN, JR., MEMBER
Water Control Branch
Engineering Division

Joseph A. McElroy

JOSEPH A. MCELROY, MEMBER
Foundation & Materials Branch
Engineering Division

Carney M. Terzian

CARNEY M. TERZIAN, CHAIRMAN
Chief, Structural Section
Design Branch
Engineering Division

APPROVAL RECOMMENDED:

Joe B. Fryar

JOE B. FRYAR
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspection. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I Investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam would necessarily represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions will be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test Flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as neccessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

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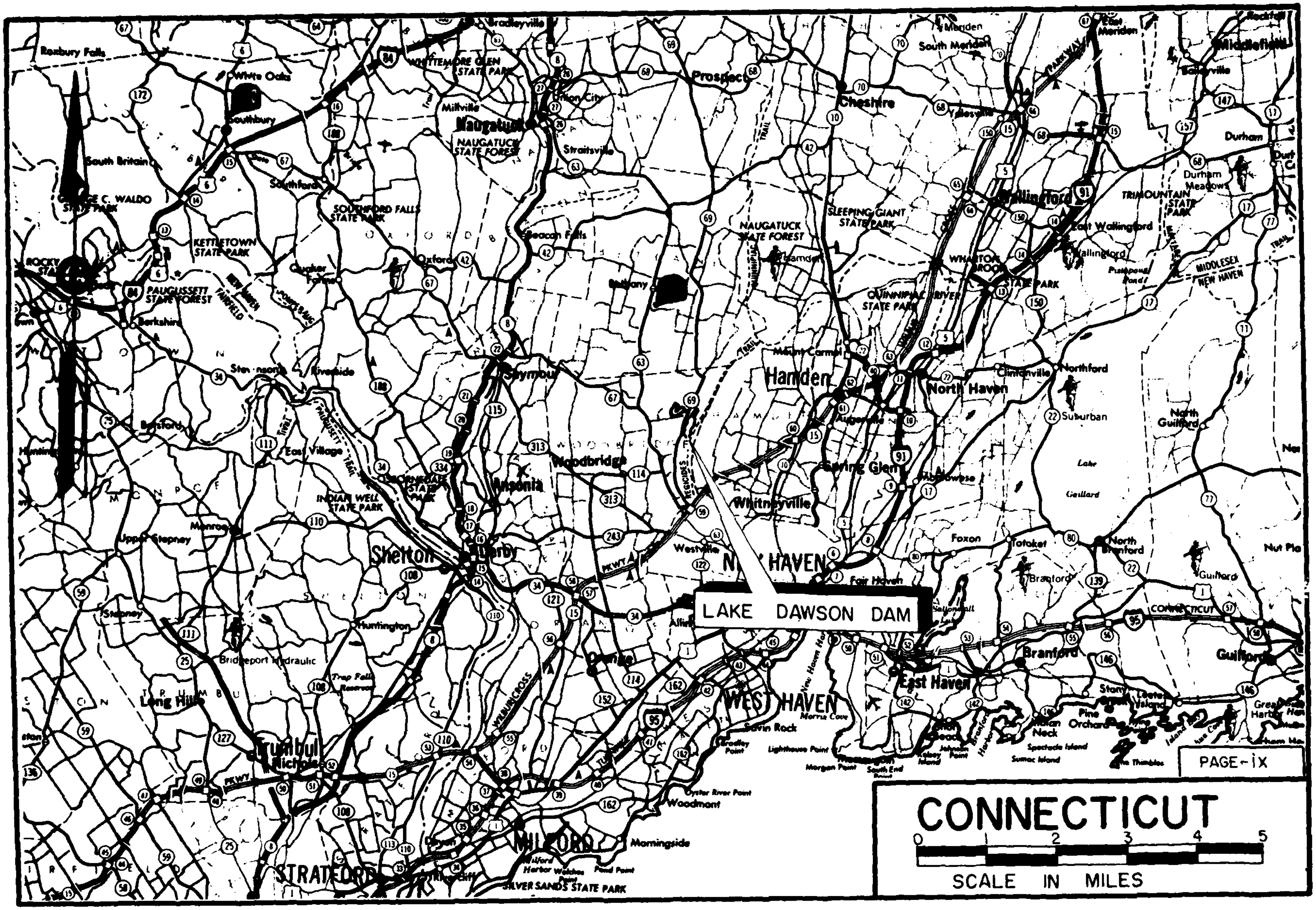
CAHN ENGINEERS INC. WALLINGFORD, CONN. ENGINEER		WEST RIVER	CONNECTICUT	CE # 27 660KA PAGE viii



LAKE DAWSON DAM
CONNECTICUT
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WEST HAVEN
MILFORD
STRATFORD
Hamden
Shelton
Derby
Ansonia
Naugatuck
Cheshire
Wallingford
North Haven
East Haven
Branford
Guilford
Woodbridge
Seymour
Orange
Trumbull
Prospect
Middlefield

PHASE I INSPECTION REPORT

LAKE DAWSON DAM

SECTION I - PROJECT INFORMATION

1.1 GENERAL

a. Authority - Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a National Program of Dam Inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Cahn Engineers, Inc. has been retained by the New England Division to inspect and report on selected dams in the State of Connecticut. Authorization and notice to proceed were issued to Cahn Engineers, Inc. under a letter of March 30, 1979 from John P. Chandler, Colonel, Corps of Engineers. Contract No. DACW 33-79-3-0059 has been assigned by the Corps of Engineers for this work.

b. Purpose of Inspection Program - The purposes of the program are to:

1. Perform technical inspection and evaluation of non-federal dams to identify conditions requiring correction in a timely manner by non-federal interests.

2. Encourage and prepare the States to quickly initiate effective dam inspection programs for non-federal dams.

3. To update, verify and complete the National Inventory of Dams.

c. Scope of Inspection Program - The scope of this Phase I inspection report includes:

1. Gathering, reviewing and presenting all available data as can be obtained from the owners, previous owners, the state and other associated parties.

2. A field inspection of the facility detailing the visual condition of the dam, embankments and appurtenant structures.

3. Computations concerning the hydraulics and hydrology of the facility and its relationship to the calculated flood through the existing spillway.

4. An assessment of the condition of the facility and corrective measures required.

It should be noted that this report does not pass judgement on the safety or stability of the dam other than on a visual basis. The inspection is to identify those features of the dam which need corrective action and/or further study.

1.2 DESCRIPTION OF PROJECT

a. Location - The dam is located on the West River in a rural section of the Town of Woodbridge, County of New Haven, State of Connecticut. The dam is shown on the New Haven U.S.G.S. Quandrangle Map as having coordinates latitude N 40° 20.0' and longitude W 72° 58.7'.

b. Description of Dam and Appurtenances - The dam is 960± feet long and its top width is 18 feet. The top of the dam is 48 feet above the bed of the West River. Construction is of earth fill with a concrete core wall. Both upstream and downstream faces are at a slope of 2 horizontal to 1 vertical. The upstream face of the dam is covered with hand placed rounded riprap to within 3 feet of the crest of the dam. There is an upstream gate house near the right end of the dam which contains all regulating outlets. At the downstream toe, directly opposite the gate house, is the original gate house, no longer used as such, which now houses an emergency power generating system. Also downstream from the toe are 2 buildings housing water treatment facilities and a fish filter system. A 6 inch tile drain runs along the toe for 185± feet, to the left of the outlet works.

The spillway is a broad-crested compound weir which spills into a concrete channel, the configuration of which is shown on Sheet B-1. The upstream approach channel is on a shallow inclination and is covered with trap-rock riprap. The spillway discharge channel is a concrete lined sloping channel as shown on Sheet B-1, and is keyed into bedrock. There are holes in the crest to install flashboards, but the operator indicated they had failed during this past winter and there were no plans to replace them. The outlets, all gated at the upstream gate house, consist of two 36 inch cast iron low level outlets, a 36 inch supply main to the treatment plant facilities fed from a wet well in the gate house, and a 6 inch

well drain. The wet well is fed by 4 intake windows, gated at the upstream face of the gate house. All gates are manually operated and all are in operable condition. Also in the gate house is a new electric pump which will be used to pump water to a treatment plant, now under construction, at the upstream end of the lake.

c. Size Classification - INTERMEDIATE - The dam impounds 1080 acre-feet of water with the lake level at the top of the earth embankment dam, which at elevation 166.8 is 48 feet above the old stream bed. According to the Recommended Guidelines, this dam is classified as intermediate in size.

d. Hazard Classification - HIGH - Immediately downstream of Lake Dawson Dam is the water filtration plant and one house. Approximately one mile downstream from the dam is Konolds Pond, along the shoreline of which are at least 10 low lying residential structures which would be in the path of rapidly rising flood waters due to a breach of Lake Dawson Dam. The flood waters would overtop Konolds Pond Dam by approximately 8 feet, subsequently discharging in the downstream channel which runs through the heavily urbanized Westville area of New Haven, which has many low lying residences.

e. Ownership - New Haven Water Company
90 Sargent Drive
New Haven, CT 06511
Mr. Jack Reynolds (203) 624-6671

f. Operator - Mr. Ken Seaton
New Haven Water Company
(203) 393-1619

g. Purpose of Dam - Public Water Supply.

h. Design and Construction History - The following information is believed to be accurate based on the plans and correspondence available. The dam was constructed in 1889-1890 by the New Haven Water Company, as engineered by Lucian A. Taylor. In 1919-1920 an upstream gate house was built by the New Haven Water Company to replace the original downstream gate house. The engineer was Albert B. Hill. In 1968-1969, the spillway was widened and lowered, and a new drain system was installed along the downstream toe. This was engineered by Malcolm Pirnie, Engineers and constructed by the Brunalli Construction Company.

i. Normal Operational Procedures - The main supply outlet is opened as needed for water supply purposes. The various level inlet gates are opened as needed to maintain water quality, based on samples taken from different levels of the lake. One low level outlet is partially opened during the dry season to provide water for the river. The low level outlets are opened for flushing for several hours once a year. During the inspection, one low level outlet was fully opened to empty the lake in order to facilitate the construction of the new treatment plant at the head of the lake. The operator reported the lake will be drained for an indeterminate length of time, most likely about 2 years.

1.3 PERTINENT DATA

a. Drainage Area - 13.4 square miles of rolling, wooded terrain, of which 0.8 square miles drains directly to Lake Dawson, and 12.6 square miles drains to upstream lakes which feed Lake Dawson. Dams whose drainage areas contribute to that of Lake Dawson include Lake Chamberlain and Glen Lake Dams on the Sargent River, and Lake Bethany, Saw Mill Pond Dam, and Lake Watrous Dam on the West River.

b. Discharge at Damsite Discharge from the lake is through a 36 inch supply main, two 36 inch low level outlets and an 6" inch well drain.

1. Outlets Works (Conduits):	2-36 inch low level outlet pipes at invert el. 118.8±
	1-36 inch main supply pipe at invert el. 126.3
	1-6 inch well drain
2. Maximum known flood @ damsite:	2100 to 2300 cfs during Oct. 16, 1955 flood. (Existing information)
3. Ungated spillway capacity @ top of dam el. 166.8:	9900 cfs
4. Ungated spillway capacity @ test flood el.:	N/A
5. Gated spillway capacity @ normal pool el:	N/A
6. Gated spillway capacity @ test flood el.:	N/A

7. Total spillway capacity @ test flood el.: N/A

8. Total project discharge @ test flood el. 168.5: 19,000 cfs

c. Elevations (Feet Above Mean Sea Level)

1. Streambed at center-line of dam: 119.3±

2. Maximum tailwater: N/A

3. Upstream portal invert diversion tunnel: N/A

4. Recreation pool: N/A

5. Full flood control pool: N/A

6. Spillway crest: 158.3

7. Design surcharge (original design): N/A

8. Top of dam: 166.8

9. Test flood design surcharge: 168.5

d. Reservoir

1. Length of maximum pool: 3500± ft.

2. Length of recreation pool: N/A

3. Length of flood control pool: N/A

e. Storage

1. Recreation pool: N/A

2. Flood control pool: N/A

3. Spillway crest pool: 1080 acre-ft.

4. Top of dam: 1540 acre-ft.

5. Test flood pool: 1670± acre-ft.

f. Reservoir Surface

1. Recreation pool: N/A

2. Flood control pool: N/A

3. Spillway crest: | 61.1 acres

3. Spillway crest:	61.1 acres
4. Test flood pool:	76 + acres
5. Top of dam:	76 acres (estimated)

g. <u>Dam</u>

1. Type:	Earth fill with concrete corewall
2. Length:	960$\pm$ ft.
3. Height:	48 ft.
4. Top width:	18 ft.
5. Side slopes:	2H to 1V (Upstream) 2H to 1V (Downstream)
6. Zoning:	N/A
7. Impervious Core:	Concrete Corewall
8. Cutoff:	N/A
9. Grout curtain:	N/A
10. Other:	N/A

h. <u>Diversion and Regulating Tunnel</u> - N/A

i. <u>Spillways</u>

1. Type:	Broad-crested compound concrete weir
2. Length of weir:	110 ft.
3. Crest elevation:	158.3
4. Gates:	N/A
5. Upstream Channel:	Shallow slope. Trap rock riprap.
6. Downstream Channel:	Concrete near-horizontal with one vertical 4' step.
7. General:	Vertical-sided concrete channel curves right and narrows to 50 ft. in width

j. <u>Regulating Outlets</u>

1. Invert:	$118.8^{\pm}$
2. Size:	2-36 inch
3. Description:	2 cast iron low level outlets.
4. Control Mechanism:	Hand operated sluice gates at upstream gate house
5. Other:	N/A

SECTION 2: ENGINEERING DATA

2.1 DESIGN

a. Available Data - The available data consists of drawings, correspondence, records, and specifications by the New Haven Water Company, the Connecticut D.E.P., Joseph W. Cone, and Malcolm Pirnie Engineers.

b. Design Features - The drawings, correspondence, records and specifications indicate the design features stated previously herein.

c. Design Data - There were no engineering values, assumptions, test results, or calculations available for the original construction. Design data for the 1968-1969 lowering of the spillway is included in Appendix B.

2.2 CONSTRUCTION

a. Available Data - The available construction drawings consist of a set of plans entitled "Lake Dawson Dam-Spillway Modifications", by Malcolm Pirnie Engineers, dated June, 1968.

b. Construction Considerations - Record drawings are available for the 1969 reconstruction of the spillway.

2.3 OPERATIONS

Lake level readings are taken daily. To our knowledge, the dam spillway capacity has never been exceeded. No other formal operating procedures are known to exist.

2.4 EVALUATION

a. Availabilty - Existing data was provided by the State of Connecticut and by the owner. The owner made the facility available for visual inspection.

b. Adequacy - Due in part to the uncertainty as to the actual location, composition, and depth of the concrete corewall, there was not sufficient detailed engineering information to perform an in-depth evaluation of the dam. Therefore, the Phase I assessment of this dam must be based primarily on visual inspection, performance history, hydraulic computations, and approximate hydrologic judgements.

c. Validity - A comparison of record data and visual observations revealed no observable significant discrepancies in the record data.

SECTION 3: VISUAL INSPECTION

3.1 Findings

a. General - The general condition of the dam is good, however, inspection did reveal some areas requiring attention. The reservoir water level was at elevation 156, 10.8 feet below the top of the dam, at the time of our inspection.

b. Dam:

Crest - The crest of the dam is a grassed earth embankment which showed no signs of cracking, settlement or subsidence.

Upstream Slope - The upstream slope is covered with hand placed riprap and is generally in good condition as shown in Photo 1. Riprap is rounded, and 12 inches or less in diameter.

Downstream Slope - A view of the downstream slope in Photo 2 shows it to be covered with sparse vegetation. In general, its condition is good and it appears to be stable. No visible signs of seepage were discovered on the downstream slope. In an area approximately 75 feet from the inlet structure, there were observed two mounds of soil with very sparse grasscover at 1/3 and 2/3 of the way up from toe of the dam, respectively. It is possible these mounds were formed after the installation of new drain pipes during 1969; they do not appear to present a problem to the dam.

The toe of the dam is covered with grass (Photo 2). One wet spot was observed at a central part of the toe of the dam and a second one was noted along the bank of the low level outlet discharge channel. The operator reported that the central area was dried up two or three years ago by installation of a 4 inch PVC drain pipe from near the toe of the dam, and a 12 inch cast iron pipe from the drain pipe to the low level outlet discharge channel. At the intersection of both pipes the water level was 4 to 6 inches below the ground elevation (See Photo 2), and the discharge from the 12 inch cast iron pipe was approximately 60 to 100 gallons per minute at the time of our initial inspection. There is a seepage exit point near the left bank of the low level discharge channel, approximately 30 to 40 feet upstream of the steel drain outlet, with a flow rate of 0.2 to 0.5± gallons per minute at the time of our initial inspection (Photo 6). A subsequent inspection 2 to 3 weeks later when the reservoir was drained, indicated only a soft wet area with no flow evident from the seep, nor from the 12 inch cast iron drain pipe further downstream.

Spillway - The concrete spillway and concrete spillway channel are generally in good condition. In the inspection period the spillway was dry and the reservoir water level was 27 inches below the spillway crest. The flashboards failed and were taken out in January of 1979 and never replaced, as reported by the operator. The concrete training walls have some cracking with leaking expansion joints in the left wall. The spillway channel slab has a central longitudinal crack with seepage from the bottom of the slab (See Photo 9). Concrete deterioration is located along the crack, caused perhaps by freeze-thaw cycles.

Discharge of approximately 4 to 6 gal./min. from both 6 inch drain pipes from beneath the channel slab was observed. A slight seepage flow was noticed also from under the right side of the slab. There is a substantial 1± foot deep wash-out under the left side of the end of the slab (Photo 10). This was most likely caused by heavy flows passing over the spillway during past years. However, there was no cracking in the concrete observed at that location on the spillway channel.

Several leaks with efflorescence are located at the left stone masonry retaining wall adjacent to the concrete spillway channel along mortar joints and in drain openings (Photo 8). These could be caused by both seepage from the reservoir and by natural groundwater.

c. Appurtenant Structures - The concrete intake chamber of the upper gate house, the new concrete service bridge and the low level outlet concrete headwall (Photo 5) are in good condition, with no evidences of significant cracks or spalling.

d. Reservoir Area - The reservoir area is bordered on the west by Conn. Route 69. The area surrounding the reservoir is wooded and largely undeveloped. Recently the New Haven water Company started construction of a new treatment plant at the extreme upstream end of the reservoir, which required partial draining of the reservoir. The operator reported that this work would be completed over a two year period.

e. Downstream Channel - The downstream spillway discharge channel runs into a channel cut into natural soil and rock formations. The natural channel bottom is covered with various sizes of boulders and cobbles (Photo 3). The channel banks are steep and have been eroded in some places. Three seepage springs with discharges from 0.5± gal./min. to 5± gal./min were discovered on the rock exposures of the downstream spillway channel right bank at a distance of 150 to 200 feet from the end of the concrete spillway (Photo 4).

3.2 Evaluations

Based upon the visual inspection, the dam appears to be in generally good condition. The following features which could influence the future condition and/or stability of the dam were identified:

1. Wet areas at the downstream toe of the dam, the water flowing from the 12 inch cast iron drain pipe from one of the wet areas, and the water level at the intersection of the perforated concrete and cast iron pipes could be indications of seepage through the dam which could worsen, especially under a full reservoir condition.

2. The new drainage system of the dam installed in 1968 and 1969 should be inspected at the manhole, and its condition should be evaluated by maintenance personnel. The seepage rate at the outlet of the drain pipe should be monitored, as excessive flow could indicate an unsafe condition in the dam.

3. Leaking cracks in the concrete training walls and the spillway slab and channel should be sealed and the concrete deterioration (cracks, potholes) should be repaired to prevent a further, more serious deterioration of concrete.

4. The wash-out under the end of the concrete spillway slab could lead to serious deterioration of the concrete slab.

5. Uplift water pressure in the foundation of the concrete slab of the spillway channel could also damage the slab if not properly relieved.

SECTION 4: OPERATIONAL PROCEDURES

4.1 REGULATING PROCEDURES

Regulating procedures consist of operating the main supply gate and inlet windows as needed for water supply purposes. One low level outlet is partially opened during dry periods to provide water downstream. Lake level readings are taken daily.

4.2 MAINTENANCE OF DAM

The downstream face of the dam has a cover of vegetation which is somewhat sparse in some areas. Grass is cut regularly and brush is cut twice per year. In addition, the downstream area is sometimes grazed by sheep. Three years ago, the New Haven Water Company instituted a yearly program of inspection of all their dams, including Lake Dawson Dam, by a consultant experienced in the field of dam inspections.

4.3 MAINTENANCE OF OPERATING FACILITIES

Maintenance of operating facilities is on an as-needed basis. Gate operating mechanisms are greased several times per year. The low level outlets are opened once a year for several hours for flushing.

4.4 *DESCRIPTION OF ANY FORMAL WARNING SYSTEM IN EFFECT*

No formal warning system is in effect. Emergencies are reported to the New Haven Water Company office.

4.5 EVALUATION

The operations and maintenance procedures are generally good, however, there are some areas requiring improvement. A formal program of operations and maintenance procedures should be implemented, including documentation to provide complete records for future reference. A formal warning system should be developed and implemented within the time frame indicated in Section 7.1c. Remedial operations and maintenance recommendations are presented in Section 7.

SECTION 5: HYDRAULIC/HYDROLOGIC

5.1 EVALUATION OF FEATURES

a. General - Lake Dawson is used as a storage reservoir for water supply, however the dam is a high spillage - low storage project, as the reservoir storage effect on the PMF inflow/outflow is minimal. Although the majority of inflow to Lake Dawson is from the outflow of Glen Lake and Lake Watrous, two upstream water company reservoirs, the regulation of the PMF by these two reservoirs is minimal and their effect on the PMF peak inflow to Lake Dawson has been neglected in our computations.

The broad-crested concrete spillway has provisions for the installation of flashboards. The flashboards, designed to fail under two feet of head above them, were in place until the storms of January 1979, at which time they collapsed. Although the owner has no plans to reinstall the flashboards, some attention was given in our computations to the hydraulic conditions during the PMF event, both with, and without the flashboards. It was concluded that operation with flashboards in place would reduce the surcharge storage capacity of the reservoir, however the spillway capacity will not actually be reduced due to the design of the flashboards for failure under two feet of head, ie. a water level 4.5 feet above the spillway crest. This flashboard design failure would result in a discharge of approximately 1000 cfs.

b. Design Data - No computations could be found for the original design of the dam. Results of hydraulic designs were available for the 1969 spillway redesign and construction, however no actual computations were obtained (See Appendix B).

c. Experience Data - No information on serious problem situations arising at the dam were found, nor has it ever been reported that the dam has been overtopped. The maximum height of water over the spillway crest was approximately 2.5 feet during the October 16, 1955 flood. It should be noted this height was prior to the redesign of the spillway to its present elevation and configuration.

d. Visual Observations - Visual inspection does not indicate that any blockage of the channel would be likely. The spillway discharge channel immediately downstream of the end of the concrete channel is a nearly vertical sided channel cut into natural soil and bedrock exposures. The channel curves to the right and, under heavy flows, will be subject to erosion along the channel bank, however serious blockage of the channel due to this erosion is not anticipated.

e. Test Flood Analysis - The test flood for this high hazard, intermediate size dam is equivalent to the Probable Maximum Flood (PMF). Based upon "Preliminary Guidance for Estimating Maximum Probable Discharge", dated March, 1978, peak inflow to the reservoir is 20,100 cfs (Appendix D-2); peak outflow is 19,000 cfs with the dam overtopped 1.7 feet (Appendix D-10). Based upon our hydraulics computations, the spillway capacity is 9,900 cfs, which is equivalent to approximately 52% of the routed Test Flood outflow.

f. Dam Failure Analysis - Utilizing the April, 1978, "Rule of Thumb Guidance for Estimating Downstream Dam Failure Hydrographs", the peak failure outflow from the dam breaching would be 133,100 cfs. A breach of the dam would result in a 20 foot high wave immediately downstream of the dam at the house and filtration plant. The breach outflow will enter Konolds Pond approximately one mile downstream rapidly raising the water level and causing Konolds Pond Dam to be overtopped by approximately 8 feet. The high water level in Konolds Pond will affect at least 10 residences along the shoreline upstream of the dam, while the outflow of about 30,000 cfs from Konolds Pond would be discharged into the heavily urbanized area of Westville immediately downstream.

SECTION 6: STRUCTURAL STABILITY

6.1 Evaluation of Structural Stability

a. Visual Observations - The visual inspection did not disclose any indications of stability problems. The inspection revealed:

1. Seepage flows probably through the lower zones and foundation of the dam which caused the wet areas downstream of the toe.

2. Seepage flows under the concrete spillway and, possibly, through the left abutment adjacent to the spillway.

The seepage could lead to a decrease of the dam reliability in the future and at present it could cause a lessening of the stability of the spillway structure (uplift pressure) and accelerated deterioration of the spillway by freezing - thawing cycles of the concrete.

b. Design and Construction Data - The design and construction data is not sufficient to permit an in-depth analysis of the stability of the dam. Data available does not include information concerning the dam cross-sections such as locations or configurations of the concrete or masonry corewall, nor does it contain information on the properties of the foundation.

c. Operating Records - The operating records do not include any indications of dam instability since its construction in 1890.

d. Post Construction Changes - The post construction changes consist of:

1. Construction of the intake gate house in 1920.

2. Construction of the new wider and larger concrete spillway and spillway channel in 1969 to provide capacity to pass a storm in excess of the Westfield, Massachusetts storm of 1955.

3. The material excavated during the lowering of the spillway was placed adjacent to the left downstream toe of the dam and to the right of the spillway discharge channel.

4. Construction of the fill berm placed at the downstream slope adjacent to the lower gate house and installation into the berm of the drain pipe system with a manhole in 1969.

The new gate house and spillway construction yield increases in the dam stability normally associated with increased discharge capacity. Placing excavated material from the spillway along the downstream toe will increase the cross-section and hence the stability of the structure, provided it does not cause a build-up of hydrostatic head within the dam, which will depend upon the permeability of the natural on-site material placed. The effect of the berm at the right end is similar; in this case, the toe drain effectively provides an outlet for seepage, thus reducing hydrostatic pressures within the dam.

e. <u>Seismic Stability</u> - The dam is in Seismic Zone 1 and according to the Recommended Guidelines, need not be evaluated for seismic stability.

SECTION 7: ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES

7.1 Dam Assessment

a. Condition - Based upon the visual inspection of the site and past performance, the dam appears to be in good condition. No evidence of structural instability was observed in the dam or its appurtenant structures. There are some areas requiring attention including the seepage problems under the embankment and the concrete spillway. Recommendations and remedial measures are presented in Sections 7.2 and 7.3, respectively.

Based upon the "Preliminary Guidance for Estimating Maximum Probable Discharges" dated March, 1978, peak inflow to the reservoir is 20,100 cubic feet per second; peak outflow is 19,000 cubic feet per second with the dam overtopped 1.7 feet. Based upon our hydraulics computations, the spillway capacity is 9,900 cubic feet per second, which is equivalent to approximately 52 percent of the routed test flood outflow.

b. Adequacy of Information - The information available is such that an assessment of the condition and stability of the dam must be based solely on visual inspection, the past performance of the dam, and sound engineering judgement.

c. Urgency - It is recommended that the measures presented in Section 7.2 and 7.3 be implemented within two years of the owner's receipt of this report.

d. Need for Additional Information - There is a need for more information as recommended in Section 7.2.

7.2 Recommendations

A registered professional engineer qualified in dam design, hydraulics and inspection should undertake further studies in four areas pertaining to the hydraulics of the dam.

a. Based upon the computations in Appendix D, the dam spillway capacity will be exceeded by the Test Flood. The owner should consider initiating more sophisticated flood routing by hydrologists/hydraulics engineers to refine the spillway design flood figures. A study should be undertaken to determine more accurately the spillway capacity and overtopping potential. Recommendations should be made by the engineer, and implemented by the owner, to increase the project discharge capacity based upon the more sopisticated hydraulic/hydrologic study.

b. Seepage emanating from near the toe of the dam, both at the wet spots near the right toe of the dam, and the seeps from along the sides of the spillway channel embankments at

the left end of the dam, should be monitored during dry periods with high and low water conditions in the reservoir. An assessment should be made of the origin and significance of all the seeps with respect to fluctuations in the reservoir water level, and if deemed necessary, recommendations should be made for the control or elimination of some or all of the seepage. The assessment of the seepage should attempt to evaluate the permeability of the dam, its foundation, the spillway base, and the left abutment by determining relative amounts of seepage through each.

c. An assessment should be made of the uplift pressure periodically being exerted on the concrete spillway channel slab. The adequacy of present measures designed to relieve the uplift pressure should be examined, and if needed, recommendations should be made to further reduce uplift pressures and to prevent seepage from surfacing through the concrete slab.

d. A program should be developed to repair leaking expansion joints and cracks in the concrete spillway and spillway slab.

7.3 Remedial Measures

a. Operation and Maintenance Procedures - The following measures should be undertaken within time frame indicated in Section 7.1c, and continued on a regular basis where applicable.

1. Round-the-clock surveillance should be provided by the owner during periods of unusually heavy precipitation. The owner should develop a formal warning system with local officials for alerting downstream residents in case of an emergency.

2. A formal program of operation and maintenance procedures should be instituted and fully documented to provide accurate records for future reference.

3. The New Haven Water Company has instituted a yearly program for inspection of all their dams, including Lake Dawson Dam, by a consultant competent in the field of dam inspection. This program, which has been in effect for the past three years, should be continued in the future on a technical basis and should include the operation of the low level outlets.

4. The wash-out under the spillway channel slab end should be filled with suitable material and compacted to increase the stability of this zone. Areas of settlement or erosion behind the spillway training walls should also be filled in.

7.4 Alternatives

This study has identified no practical alternatives to the above recommendations.

APPENDIX A

INSPECTION CHECKLIST

VISUAL INSPECTION CHECK LIST
PARTY ORGANIZATION

PROJECT Lake Dawson Dam

DATE: May 1, 1979

TIME: 9:20 a.m.

WEATHER: sunny, 70°

W.S. ELEV.____U.S.____DN.S

PARTY:	INITIALS:	DISCIPLINE:
1. Peter Heynen	PMH	Cahn Engineers, Inc
2. Calvin Goldsmith	CRG	Cahn Engineers, Inc
3. Miron Petrovsky	MP	Cahn Engineers, Inc
4. George Stephens	GS	Cahn Engineers, Inc
5. Al Bucher	AB	New Haven Water Co.
6.		

PROJECT FEATURE	INSPECTED BY	REMARKS
1. Earth Dam	PMH, CRG, MP, GS, AB	
2. U/S Gate House w/Service Bridge	" " " " "	
3. Low Level Outlet	" " " " "	
4. Spillway	" " " " "	
5. Downstream Channel	" " " " "	
6.		
7.		
8.		
9.		
10.		
11.		
12.		

PERIODIC INSPECTION CHECK LIST

Page A-2

PROJECT LAKE DAWSON DAM

DATE MAY 1, 1979

PROJECT FEATURE EARTH DAM EMBANKMENT BY PMH, CRG, MP, GS, AB

AREA EVALUATED		CONDITION
DAM EMBANKMENT		
Crest Elevation		65.3
Current Pool Elevation		
Maximum Impoundment to Date		N/A
Surface Cracks		NONE OBSERVED
Pavement Condition		NO PAVEMENT
Movement or Settlement of Crest		NONE OBSERVED
Lateral Movement		NONE OBSERVED
Vertical Alignment		APPEARS GOOD
Horizontal Alignment		APPEARS GOOD
Condition at Abutment and at Concrete Structures		GOOD - NO APPERENT EROSION
Indications of Movement of Structural Items on Slopes		NONE OBSERVED
Trespassing on Slopes		NONE
Sloughing or Erosion of Slopes or Abutments		NONE OBSERVED
Rock Slope Protection-Riprap Failures		GOOD CONDITION
Unusual Movement or Cracking at or Near Toes		NONE OBSERVED
Unusual Embankment or Downstream Seepage		WET AREAS AT DAM TOE AND SUBSTANTIAL SEEPAGE FROM 6" STEEL DRAIN PIPE
Piping or Boils		NONE OBSERVED
Foundation Drainage Features		NONE KNOWN
Toe Drains		
Instrumentation System		NONE KNOWN

PERIODIC INSPECTION CHECK LIST

Page A-3

PROJECT Lake Dawson Dam

DATE May 1, 1979

PROJECT FEATURE ____________

BY RVH, CRG, MD, GS, AB

AREA EVALUATED		CONDITION
<u>OUTLET WORKS-INTAKE CHANNEL AND INTAKE STRUCTURE</u>		
a) <u>Approach Channel</u>		
Slope Conditions		not observed
Bottom Conditions		
Rock Slides or Falls		N/A
Log Boom		
Debris		none observed
Condition of Concrete Lining		N/A
Drains or Weep Holes		not observed
b) <u>Intake Structure</u>		
Condition of Concrete		good
Stop Logs and Slots		not observed

PERIODIC INSPECTION CHECK LIST

Page 4-4

PROJECT LAKE DAWSON DAM

DATE MAY 1979

PROJECT FEATURE OUTLET CONTROL TOWER

BY SYH, CRG, MP, CS, AB

AREA EVALUATED		CONDITION
OUTLET WORKS-CONTROL TOWER		
a) Concrete and Structural		BRICK HOUSE AND CONCRETE CHAMBER
General Condition		GOOD
Condition of Joints		NONE OBSERVED
Spalling		NOT OBSERVED
Visible Reinforcing		NONE OBSERVED
Rusting or Staining of Concrete		NOT OBSERVED
Any Seepage or Efflorescence		NOT OBSERVED
Joint Alignment		NOT OBSERVED
Unusual Seepage or Leaks in Gate Chamber		NOT OBSERVED
Cracks		NONE OBSERVED
Rusting or Corrosion of Steel		NOT OBSERVED
b) Mechanical and Electrical		
Air Vents		
Float Wells		
Crane Hoist		
Elevator		
Hydraulic System		
Service Gates		[illegible]
Emergency Gates		NONE
Lightning Protection System		
Emergency Power System		
Wiring and Lighting System		

PERIODIC INSPECTION CHECK LIST

PROJECT Lake Dawson Dam DATE May [illegible]

PROJECT FEATURE Low Level Outlet BY [illegible], A2

AREA EVALUATED		CONDITION
OUTLET WORKS-OUTLET STRUCTURE AND OUTLET CHANNEL		
General Condition of Concrete		CONCRETE HEADWALL - GOOD
Rust or Staining		NONE
Spalling		SOME
Erosion or Cavitation		NONE
Visible Reinforcing		NONE
Any Seepage or Efflorescence		MINOR
Condition at Joints		NONE
Drain Holes		NONE
Channel		
Loose Rock or Trees Overhanging Channel		NONE
Condition of Discharge Channel		BEDROCK & GRAVEL BOTTOM - GOOD CONDITION

PERIODIC INSPECTION CHECK LIST

Page A-[illegible]

PROJECT LAKE DAWSON DAM DATE [illegible]

PROJECT FEATURE SPILLWAY AND CHANNELS BY [illegible] AB

AREA EVALUATED	CONDITION
OUTLET WORKS-SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS	
a) Approach Channel	
General Condition	GOOD
Loose Rock Overhanging Channel	NONE
Trees Overhanging Channel	NONE
Floor of Approach Channel	GOOD
b) Weir and Training Walls	
General Condition of Concrete	GOOD
Rust or Staining	NONE
Spalling	SOME CRACKING AND SPALLING IN TRAINING WALLS AND BOTTOM OF TRANSITION SECTION
Any Visible Reinforcing	NONE
Any Seepage of Efflorescence	SEEPAGE THROUGH CRACKS IN BOTTOM OF SLAB
Drain Holes	Good, SUBSTANTIAL [illegible] FLOW
c) Discharge Channel	
General Condition	GOOD - NATURAL BEDROCK
Loose Rock Overhanging Channel	NONE IN CHANNEL
Trees Overhanging Channel	NONE IN CHANNEL
Floor of Channel	BEDROCK & BOULDERS
Other Obstructions	[illegible] ERODED SPOTS

PERIODIC INSPECTION CHECK LIST

Page A-7

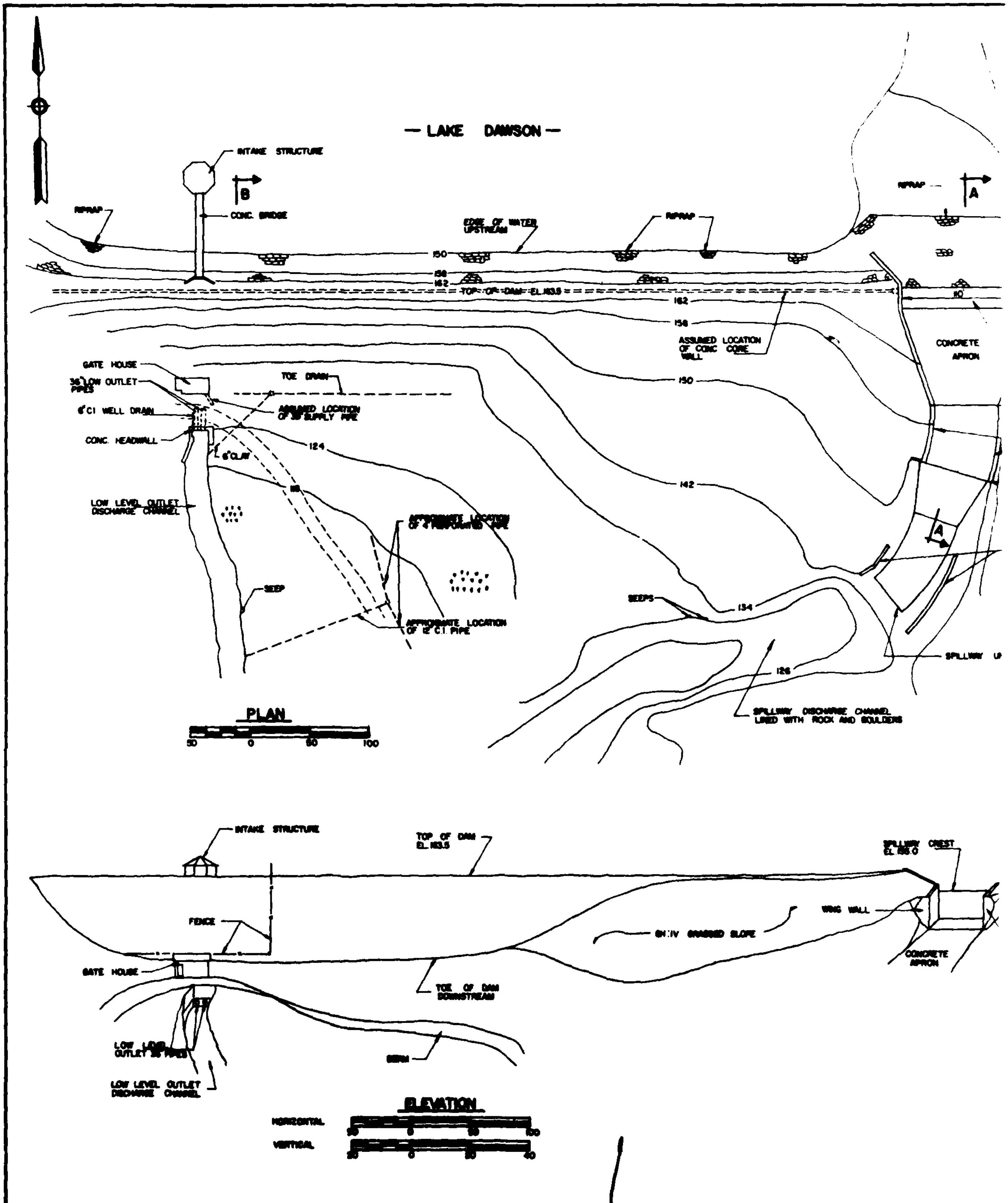
PROJECT LAKE DAWSON DAM　　DATE MAY 1, 1979

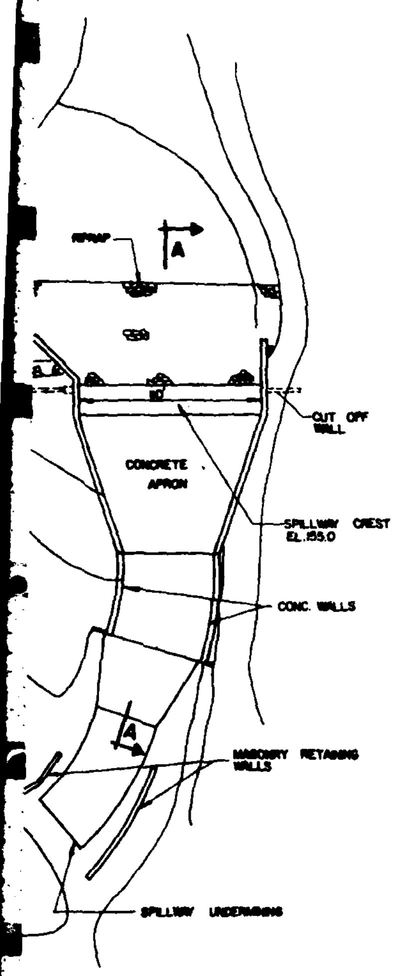
PROJECT FEATURE CONCRETE BRIDGE　　BY CMH, CRG, MP, GB, JB

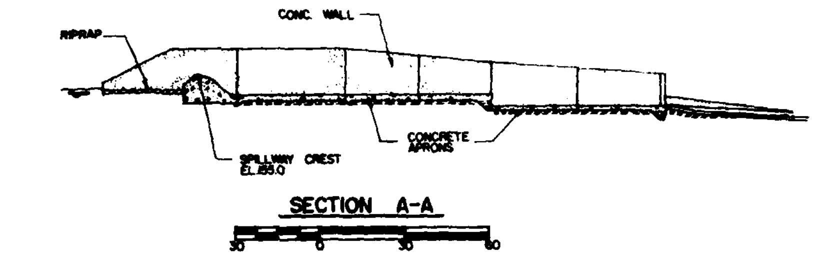
AREA EVALUATED	CONDITION
OUTLET WORKS-SERVICE BRIDGE	BRIDGE BETWEEN UPPER GATE HOUSE AND EMBANKMENT
a) Super Structure	
Bearings	GOOD
Anchor Bolts	
Bridge Seat	
Longitudinal Members	
Under Side of Deck	GOOD
Secondary Bracing	
Deck	GOOD
Drainage System	N/A
Railings	
Expansion Joints	GOOD
Paint	N/A
b) Abutment & Piers	
General Condition of Concrete	GOOD
Alignment of Abutment	NOT OBSERVED
Approach to Bridge	GOOD
Condition of Seat & Backwall	

APPENDIX B

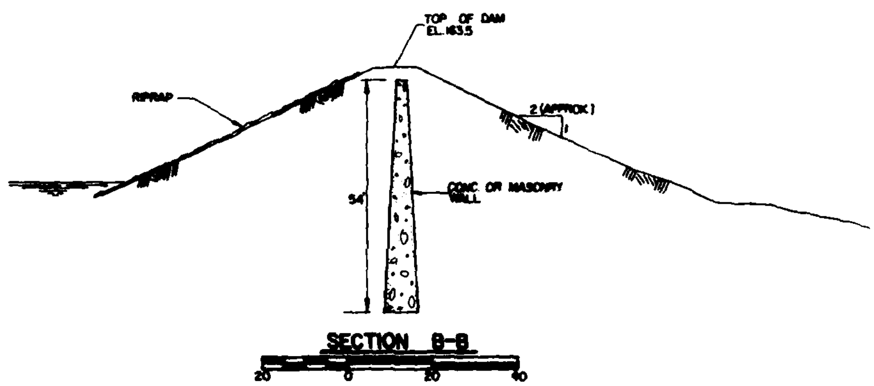
ENGINEERING DATA AND CORRESPONDENCE


— LAKE DAWSON —
INTAKE STRUCTURE
B
A
RIPRAP
CONC. BRIDGE
EDGE OF WATER UPSTREAM
TOP OF DAM EL 163.5
150
156
162
ASSUMED LOCATION OF CONC CORE WALL
CONCRETE APRON
GATE HOUSE
36" LOW OUTLET PIPES
TOE DRAIN
6" C.I. WELL DRAIN
ASSUMED LOCATION OF 36" SUPPLY PIPE
CONC. HEADWALL
6" CLAY
124
142
LOW LEVEL OUTLET DISCHARGE CHANNEL
SEEP
SEEPS
134
126
APPROXIMATE LOCATION OF 4" PERFORATED PIPE
APPROXIMATE LOCATION OF 12" C.I. PIPE
SPILLWAY DISCHARGE CHANNEL LINED WITH ROCK AND BOULDERS
PLAN
50 0 50 100
INTAKE STRUCTURE
TOP OF DAM EL 163.5
SPILLWAY CREST EL 155.0
WING WALL
FENCE
GRASSED SLOPE
CONCRETE APRON
GATE HOUSE
TOE OF DAM DOWNSTREAM
LOW LEVEL OUTLET DISCHARGE CHANNEL
BERM
ELEVATION
HORIZONTAL
50 0 50 100
VERTICAL
20 0 20 40





SECTION A-A

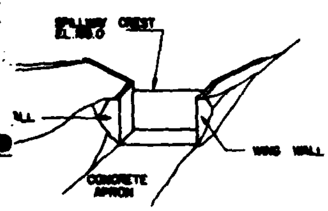


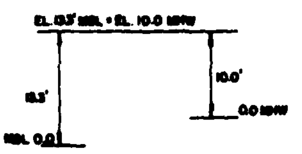
SECTION B-B

NOTES:

1. THIS PLAN WAS COMPILED FROM CAHN ENGINEERS FIELD SURVEY AND FROM A SET OF PLANS ENTITLED "LAKE DAWSON DAM SPILLWAY MODIFICATIONS" BY MALCOLM PIRNIE ENGINEERS DATED JUNE 1968 AND CONFIRMED AS A RECORD DRAWING ON JUNE 5, 1973. SOME CONFIGURATIONS SHOWN ARE APPROXIMATE IN DIMENSION, AND NOT ALL STRUCTURAL AND/OR TOPOGRAPHIC FEATURES IDENTIFIED.

2. ELEVATIONS SHOWN ARE BASED ON THE NEW HAVEN WATER COMPANY (MEAN HIGH WATER) DATUM WHICH IS 3.30 FEET ABOVE THE MEAN SEA LEVEL DATUM. A GRAPHICAL DATUM CONVERSION IS SHOWN ON THIS PLAN.





DATUM CONVERSION

CAHN ENGINEERS INC. WALLINGFORD, CONNECTICUT ENGINEER	U.S. ARMY ENGINEER DIV NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

PLAN, ELEVATION AND SECTIONS

LAKE DAWSON DAM

WEST RIVER — WOODBRIDGE, CONNECTICUT

DRAWN BY	CHECKED BY	APPROVED BY	SCALE AS NOTED	
J. N.	[illegible]	[illegible]	DATE JUNE 1979	SHEET B-1

LIST OF EXISTING PLANS

"New Haven Water Co.
West River System
Plan of Lake Dawson Gatehouse
Town of Woodbridge, Ct."
Office of Albert B. Hill, Consulting Engineer
June, 1919

"New Haven Water Co., New Haven, Conn.
Dawson Dam Spillway Modification"
6 sheets
Malcolm Pirnie Engineers
June, 1968

SUMMARY OF DATA AND CORRESPONDENCE

DATE	TO	FROM	SUBJECT	PAGE
May 19, 1964	Files	Water Resources Commission, Supervision of Dam	Inventory data	B-4
Aug. 12, 1971	Files	New Haven Water Co.	Statistics on dams	B-5
Apr. 29, 1963	A. L. Corbin, President, New Haven Water Co.	Joseph A. Novaro, Chief Engineer, New Haven Water Co.	Hydraulic data and computations on West River Watershed	B-8
Apr. 12, 1965	Joseph W. Cone, P.E.	Joseph A. Novaro	Additional hydraulic data on West River System	B-11
June 26, 1965	William P. Sander, Water Resources Commission	Joseph W. Cone, P.E.	Summary of report concerning dams owned by the New Haven Water Co. on the West and Sargent Rivers	B-12
July 15, 1966	William Wise, Director, Water Resources Commission	Joseph A. Novaro	Progress report on West River System study	B-18
Jan., 1967	New Haven Water Co.	Malcolm Pirnie Engineers	Excerpts from report on flood flows and spillway capacities of West River system with recommendations to increase spillway capacities	B-19

DATE	TO	FROM	SUBJECT	PAGE
Aug. 2, 1967	New Haven Water Co.	Malcolm Pirnie Engineers	Report on effects of maximum possible storm on West River system spillways	B-41
1967	New Haven Water Co.	Malcolm Pirnie Engineers	Summary of calculations on existing dam spillway; recommendations for modifications, with plans	B-44
July 26, 1969	Water Resources Commission	Joseph A. Novaro, Chief Engineer, New Haven Water Co.	Application for construction permit for spillway modifications	B-47
Aug. 20, 1968	Files	William H. O'Brien, III Water Resources Commission	Additional data on flashboard design	B-49
Mar. 2, 1971	Files	William H. O'Brien, III	Result of inspection of spillway; report of minor leaks	B-50
Mar. 31, 1971	Joseph A. Novaro	William H. O'Brien, III	Report of leaks; request for information on as-built condition of spillway	B-51
Apr. 7, 1971	William H. O'Brien,	Joseph A. Novaro	Data on as-built condition of spillway	B-52
July 5, 1979	Files	Cahn Engineers, Inc.	Centerline elevations at top of dam	B-53

SUPERVISION OF DAMS
INVENTORY DATA

Long 72-58.7

Lat 41-22.0

Inventoried By WPS

Date 19 MAY 1964

Name of Dam or Pond LAKE DAWSON

Code No. WS 7.9

Nearest Street Location ROUTE 69

Town WOODBRIDGE

U.S.G.S. Quad. NEW HAVEN

Name of Stream WEST RIVER

Owner NEW HAVEN WATER COMPANY

Address 100 CROWN STREET
NEW HAVEN

OK 7/73

Pond Used For WATER SUPPLY

Dimensions of Pond: Width 1000 FEET Length 3000 FEET Area ~~75~~ 69.5 ACRES

Total Length of Dam 900 FEET Length of Spillway ~~55~~ 110 FEET

Location of Spillway EAST END OF DAM

Height of Pond Above Stream Bed ~~50~~ 13 FEET

Height of Embankment Above Spillway 6 FEET

Type of Spillway Construction CONCRETE

Type of Dike Construction EARTH, RIP-RAP UPSTREAM

Downstream Conditions NEW HAVEN

789

Summary of File Data

Remarks

B-4

Would Failure Cause Damage? YES Class B

NEW HAVEN WATER COMPANY

STATISTICS ON DAMS*

NAME Dawson

SUPPLY SYSTEM West River

LOCATION Woodbridge

DATES: ORIGINAL CONSTRUCTION 1889-1890

ADDITIONS, ALTERATIONS 1919-1920; 1968-1969

	MEAN HIGH WATER ELEVATION	LENGTH
CREST**	164.0	1000± Ft.
TOP OF CORE WALL		
SPILLWAY	157.50	110 Ft.
B. O. AXIS	117.0	2@ 240 Ft. each
BED OF RIVER	116±	
DEEPEST FOUNDATION	109±	

FREEBOARD: CREST TO SPILLWAY 6.50 Ft.

CREST TO TOP OF CORE WALL

HEIGHT: CREST TO BED OF BROOK 48± Ft.

CREST TO DEEPEST FOUNDATION 55± Ft.

TYPE Earth and Concrete Corewall

TOP WIDTH--MAX. BOTTOM WIDTH (Ft.) 18± -- 200±

UPSTREAM SLOPE H/V 2/1

DOWNSTREAM SLOPE H/V 2/1

TRIBUTARY WATERSHED (Square Miles) 13.0

RESERVOIR AREA (Acres) 69.5

RESERVOIR TOTAL STORAGE (MG) 352

RESERVOIR USABLE STORAGE (MG) 237

*See individual sheets for more details
**Crest Length includes spillway

Date 8/12/74

NEW [illegible]

DATE Aug. 1977

[NA]ME OF DAM DAWSON

[TY]PE

Earth dam with concrete corewall. Corewall rests in part on rock and in part on hardpan material. Rip rap on upstream face. Original construction in 1889-1890 included a downstream gate house only. An upstream gate house was installed in 1919-1920. New wider spillway and new spillway channel installed in 1968-1969

[LO]CATION

In Woodbridge on the West River approximately two miles north and upstream from the Woodbridge - New Haven Town Line on the east side of State Highway No. 69 known as the Litchfield Turnpike. In 1889-1890 it was located as being about 3500 feet below the "Valley Mill" in Woodbridge.

[S]UPPLY SYSTEM West River

[D]ATE OF CONSTRUCTION

ORIGINAL 1889-1890 - dam with downstream gate house

OTHER 1919-1920 - upstream intake gate house constructed.

1968-1969 — new wider, larger spillway and spillway channel constructed to provide capacity to handle runoff from a 1000 year frequency storm. Additional fill placed on portions of downstream slope including a berm along a portion of downstream toe of embankment. New drain line system installed in berm at downstream toe, draining to West River.

[E]NGINEER	7. CONTRACTOR
18[8]9-1890 LUCIAN A. TAYLOR	New Haven Water Company
19[1]9-1920 ALBERT B. HILL	New Haven Water Company
19[6]8-1969 MALCOLM PIRNIE ENGINEERS	The Brunalli Construction Company

	ELEVATION	LENGTH (Feet)	MISC.
CREST	164.0 M.H.W.	± 1000	
SPILLWAY	157.50 M.H.W.	110	Top of flashboards
AXIS OF B.O	117 MHW.	±240' each	2-36" C.I. Blowoffs
BED OF RIVER	± 116 MH.W		
DEEPEST FNDN.	±109 M.H.W.		

13. HEIGHT FROM BED OF BROOK ± 48 Feet
14. HEIGHT FROM DEEPEST FOUNDATION ± 55 Feet
15. TOP WIDTH ± 18 Feet
16. MAXIMUM WIDTH AT BOTTOM ± 200 Feet
17. UPSTREAM SLOPE 2 Hor. on 1 Ver.
18. DOWNSTREAM SLOPE " " " " "

19. FREE BOARD - SPILLWAY TO CREST 6.5 Feet
- SPILLWAY TO TOP OF COREWALL 1.8 Feet

20. MISC. DATA.
* Reservoir capacity increased 28,830,707 gallons by removal and sale of 142,745 cubic yards of sand, gravel and boulders in 1966-1969 inclusive. Dawson spillway repaired in 1912 by grouting.

21. WATERSHED TRIBUTARY TO:
UPSTREAM DAMS 12.2 Sq. Mi
THIS DAM 0.8 Sq. Mi
TOTAL WATERSHED TRIBUTARY TO THIS DAM 13.0 Sq. Mi

22. RESERVOIR AREA AT FLOW LINE 69.5 Acres
23. RESERVOIR CAPACITY AT FLOW LINE * 352 Mil. Ga
24. RESERVOIR USABLE CAPACITY (To lowest outlet) Top 12' * 237 Mil. Ga

25. UPSTREAM DAMS
Chamberlain Dam & Glen Dam on Sargent River and Bethany Dam and Watrous Dam on the West River of New Haven Water Co. The privately owned Burnham Saw Mill Dam & Pond, immediately upstream from the upper end of Watrous Reservoir is part of the 3.2 Sq. Mile watershed tributary to Watrous Dam

26 DOWNSTREAM DAMS
Konold Pond Dam
Pond Lilly Dam of the Pond Lilly Company, Whalley Ave, New Haven

To: Mr. A. L. Corbin, Jr., President
From: Joseph A. Novaro, Chief Engineer

Re: West River Watershed.

Flood conditions in 1955 at and upstream from the Whalley Avenue bridge in Westville, generally attributed to the West River, actually were the result of heavy storm runoffs from several watersheds:

1. West River lying west and south of West Rock, eventually passing under the Whalley Avenue bridge.

2. An area starting at the Yale Golf Course ponds and extending north to the Fountain Street - Whalley Avenue area, draining to West River.

3. Wintergreen Brook lying east of West Rock. It enters West River about 600 feet north of the Whalley Avenue bridge.

4. Farm Brook, east of West Rock, starting about one mile north of Paradise Park in Hamden and draining south into Wintergreen brook about 1900 feet southeast of the Springside Home.

5. An unnamed brook lying between 3 and 4 above, which starts about one-half mile west of Paradise Park in Hamden and drains south into Wintergreen Brook at a point in the Brookside Housing area of New Haven.

6. Beaver Pond watershed which stretches approximately from Arch Street in Hamden south to Goffe Street in New Haven. The brook from Beaver Pond runs southwest, entering Wintergreen Brook about 900 feet north of the Whalley Avenue bridge.

The watersheds tributary to the Whalley Avenue bridge total 29.3 square miles which I have broken down, for analysis, into three main areas:

North of and tributary to Dawson Dam	13.9 sq. mi.
" " " " " Wintergreen Dam	1.5 sq. mi.
Remaining watershed	13.9 sq. mi.
Total	29.3 sq. mi.

The New Haven Water Company owns approximately one square mile of the 1.5 square miles of watershed tributary to Lake Wintergreen and about 8 square miles of the 13.9 square miles of watershed tributary to Lake Dawson. The balance is owned by others. The Company owned land, used for water supply purposes only, and well forested, has not contributed to any increase in flood runoff. In fact the Company's forestry program has effected some decrease in the rate of storm water runoff from the land.

The balance of the land owned by others and draining to the Whalley Avenue bridge has been and will continue to be developed for housing, schools, industry and colleges. Their roofs, driveways, streets and parking areas increase the amount and rate of storm water runoffs and storm water sewers, where installed, accelerate the runoff.

Our reservoirs generally start to go down early in June and continue to go down until late in the year. About half the year's refilling starts about the middle of November and about the middle of December the rest of the years. Occasionally, as recently experienced, our reservoirs start to refill in January and very occasionally in February. Our reservoirs thus are in a position during the hurricane season to receive and retain a large portion, and sometimes all, of the storm runoff from the 15.4 square miles tributary to them.

In August 1955 hurricane Connie, followed by Dianne, brought heavy rains to this area. Dianne caused considerable damage in Milford and the lower Naugatuck Valley. In the period August 8 to 14 inclusive rainfall at Lake Dawson totalled 4.14". On August 18 and 19 hurricane Dianne brought an additional 6.67 . In one 24 hour period 4.87" fell at Dawson.

In this extended storm period our reservoirs received and retained 477 million gallons of water. Glen, Watrous, Chamberlain, Bethany and Wintergreen retained all the runoff reaching them, allowing nothing to go downstream. Dawson, on August 19th, with its small tributary watershed of 0.8 square miles, finally filled but the depth of flow over the spillway was only one half an inch. The data is listed herewith:

	Before the Storms		After the Storms	
	Reservoir Level	Million gals. to fill	Reservoir Level	Million gals. to fill
Dawson	down 0' 1/2"	1	Full	0
Glen	" 21' 3"	140	down 10' 3"	81
Watrous	" 6' 4"	209	" 2' 7"	86
Chamberlain	Empty	164	" 4' 10"	49
Bethany	down 4' 6"	138	Full	0
Wintergreen	" 6' 6'	66	down 2' 5"	25
Totals		718		241

Amount retained 718 - 241 = 477 million gallons.
In addition about 9 million gallons per day throughout the entire storm period was also utilized for water supply purposes.

The heavy storm on October 14 to 17 inclusive in 1955 produced floods and considerable damage in the Westville area. Our rain gauge at Dawson registered 8.84" of rainfall in this period. Of this 5.85" fell in one 24 hour period alone. Our reservoirs were all full after this storm but prior to filling they stored and retained 251 million gallons of water as shown in the data below:

	Reservoir level	Million gals. to fill
Dawson	over 0' 1/2"	0
Glen	down 5' 1"	45
Watrous	" 2' 6"	83
Chamberlain	" 9' 4"	88
Bethany	" 0' 1/2"	2
Wintergreen	" 3' 3"	33
		251

In addition, at the height of the storm water runoff our reservoirs temporarily stored 215 million gallons additional above their spillways, preventing even higher flood levels down stream by releasing this over a

greater period of time. The data is herewith:

	Depth above Spillway	Surface Acres	Acre-feet	Million gals.
Dawson	2' 5"	71	172	55.5
Glen	2' 7"	27	70	22.6
Watrous	1' 11"	110	211	68.1
Chamberlain	2' 0"	37	74	23.9
Bethany	0' 11"	106	97	31.3
Wintergreen	1' 0"	44	44	14.1
				215.5

The effect of reservoir storage above the spillway level on downstream flood conditions can be checked by comparing the flood runoff from the reservoir-controlled watersheds with that of the other watersheds as follows:

1. From lake level records (depths on spillway) I have computed that at peak runoff approximately 1425 cubic feet per second were passing our Dawson and Wintergreen dams. For the 15.4 sq. miles of tributary watershed this is an average runoff rate of 92 cubic feet per second per square mile of watershed. (September 1938 hurricane runoffs were in the 40 to 80 range).

2. The peak flow under the Whalley Avenue bridge, computed by Consultants for the State, was 3,525 cubic feet per second. Subtracting above 1425 cusec. leaves 2,100 cusec. contributed by the remaining, uncontrolled 13.9 square miles or at an average runoff rate of 151 cubic feet per sec. per square mile.

Peak runoff rate from the uncontrolled portions of the watersheds therefore was about 50 per cent higher than from the controlled watersheds for this particular storm. This is not surprising when you consider the absence of large reservoirs and the large amount of impervious surfaces in the built up residential, commercial, school and industrial areas.

Consultants for the State reported that a 48 inch diameter sewer suspended under the floor of the bridge restricted the flow area of the bridge, accentuating flood conditions upstream. In order to pass the computed possible flood flow at this point - larger than the 3,525 cusec. - the Consultants recommended that the sewer be replaced with a siphon under West River and that additional waterway capacity be provided by widening the bridge.

Since this flood New Haven Water Company has raised Chamberlain Dam 35 feet increasing its storage from 164 million gallons to 894 million gallons. Thus in the fugure additional space has been provided to store and retain flood runoffs.

While Company owned land will remain well forested, retaining normal yield and runoff, the areas owned by others will continue to be developed for other uses - uses which will inevitably increase the amount of storm runoff and the rate of runoff.

NEW HAVEN WATER COMPANY
NEW HAVEN, CONNECTICUT 06506

Tel. MA 4-9803

April 12, 1965

Mr. Joseph W. Cone,
Civil Engineer,
124 Havemeyer Place,
Greenwich, Conn.

Dear Mr. Cone:

Referring to your letter of April 2, 1965, we enclose the following:

1. Data forms for Chamberlain, Glen, Bethany, Watrous and Dawson Dams.

2. Plans for above dams.

3. Sanitation map showing limits of watershed tributary to above dams.

In the period from 1937 to the present, depths over the spillways of the above dams in most cases have been less than one foot.

Our rain gauge at Lake Dawson recorded a total of 4.14" in the August 8 - 14, 1955 storm. It recorded 6.67" on August 18-19, 1955. In one 24-hour period rainfall totalled 4.87". None of the runoff went downstream but Lake Dawson was full at the end of the storm.

The Lake Dawson rain gauge recorded 8.84" of rain in the October 14-17, 1955 storm, or which 5.85" fell in one 24-hour period. This storm filled the four upstream reservoirs. Maximum depths on spillways occurred on October 16, 1955 and are recorded on the data forms.

Chamberlain Dam was raised in 1958-1959 and a new larger spillway was provided. Storage was increased from the original 164 million gallons to the present 894 million gallons.

If you will let me know when you wish to make a field inspection, I will be glad to make the necessary arrangements.

Yours very truly,
NEW HAVEN WATER COMPANY

Joseph A. Novaro

Joseph A. Novaro
Chief Engineer

1965

REPORT

CONCERNING DAMS

Owned by

NEW HAVEN WATER CO.

BETHANY

WATROUS

CHAMPERLAIN

GLEN

DAWSON

on the

WEST & SARGENT RIVERS

J. W. Cone P.E.
June 1965

I N D E X

Part I

Part II

NOTE: Maps, graphs, etc., are in separate folder.

...NECTICUT REGISTRATION 4 CIVIL ENGINEER TOWNSEND 8-8184
124 HAVEMEYER PLACE
GREENWICH, CONNECTICUT
06830

June 26, 1965

Mr. William P. Sander
Water Resources Commission
State Office Building
Hartford 15, Conn.

Re: Dams #35 - 1 to 5
New Haven Water Co.

Dear Mr. Sander:

First, I apologize for not completing this assignment more promptly; reasons being that a low quality virus for over a month left me with no pep mentally or physically, and delays in obtaining certain plans and information.

The assignment was- "we would like to know the present condition of these dams" - Bethany - Watrous - Dawson on West River and Chamberlain - Glen on Sargent River, a tributary to West River above Dawson Dam.

In my opinion, the "condition" of these dams is good as regards masonry of the three masonry gravity dams and the upkeep of two earth embankment dams.

But as regard to whether or not the dams are safe, particularly as regard spillway capacity, my opinion is as follows:

35-1 Bethany Spillway is inadequate. However a thin sheet over a length of 990' will do comparatively little damage except to highway. The gravity section is safe.

35-2 Watrous Generally same remarks as for Bethany.

35-3 Chamberlain Spillway is adequate in every respect as is the dam. It is reassuring to find a spillway that will carry 1525 cfs per sq. mi. on 4.1 sq. mi. Note Items #26 & 28 on Data Sheet.

35-4 Glen Spillway is nowhere near adequate. In fact, Oct. '55 flood nearly overtopped earth section at left or east abutment. Section of dam is safe.

Right abutment should be raised to protect highway.

Left abutment should be investigated:-

(a) To determine whether or not there is a core wall.

(b) Possibility of emergency spillway or fuse plug.

(c) Note Items #26 & 28 on Data Sheet.

35-5 Dawson Present spillway is entirely inadequate to carry probable floods of the present and future. In fact, the dam would have been overtopped if certain saving factors had not been present in Oct. 1955.

(a) Not an excessive rainfall, only about R of 50 yr. (Compare with precipitation graphs)

(b) Several of reservoirs were below FL (See data notes by Navaro which you have)

(c) Flood Q '55 at Dawson of about 2100 cfs has an R value 3.8 (2100 ÷ 560) equivalent to 120 yr on old Conn. curve and 55 yr on revised 1965 curve. (See graph PL 13)

Items #26 & 28 on Data Sheet are particularly illuminating.

It does not need a lively imagination to visualize what would happen to Westville and New Haven if Dawson should be overtopped; Norwich failure would be peanuts comparatively.

A brief discussion of pertinent data and situations follows. Also there are prints of sections of dams, precipitation graphs and various other graphs that I used or are pertinent to this investigation for general information or checking purposes.

Please excuse the informality and crudness of the matter submitted, the objective being to reduce costs to the minimum.

I would observe that Mr. Navaro, Mr. Ferris and Mr. Reynolds of the New Haven Water Co. were most cooperative as was Mr. Thomas of the U.S. Geological Survey.

My recommendation is that the New Haven Water Co. be advised that their consulting engineers should investigate the entire system, with particular emphasis on

conditions at Glen and Dawson, and submit corrective measures.

Yours very truly,

J. W. Cone

JWC/dr

Enc: Part II
Photos (11)

NEW HAVEN WATER COMPANY
NEW HAVEN, CONNECTICUT 06506

July 15, 1966

Mr. William Wise, Director,
Water Resources Commission,
State Office Building,
Hartford 15, Conn.

Dear Mr. Wise:

As promised we are writing to report progress to date on the studies of our West River System.

Our consultants, Malcolm Pirnie Engineers, have gathered all available data concerning the 1955 hurricane storms and the characteristics of the West River and Sargent River watersheds, reservoirs, and dams. This information has been supplemented by a field investigation by them.

They are using the unit hydrograph method of analysis. Their first step is to reconstruct one of the 1955 storms and route it through the watersheds. If, by this procedure, they can produce, within reason, the conditions which were observed at the various dams during the 1955 storms, the characteristics of the unit hydrograph and the procedure can be considered verified.

With the procedure verified, they plan to route a 100-year storm and a 1000-year storm through the reservoir systems. The results of these runs will be used to determine what improvements to recommend. Stability analyses will be made after the design hydraulic conditions have been determined.

To date our consultants have completed their general hydrologic investigations; have constructed unit-hydrographs to be used with the drainage areas tributary to each dam and reservoir; have selected and arranged rainfall data to be used for the 1955 storm and for the 100-year and 1000-year storms and have computed in-flow hydrographs into each of the reservoirs for the 1955 storms. Rating curves are being computed for each spillway. When these computations are completed the 1955 storm will be routed through the system in order to verify the procedure.

Our consultants advise that their final report should be ready by the end of September.

Yours very truly,
NEW HAVEN WATER COMPANY

Joseph A. Novaro

Joseph A. Novaro
Chief Engineer

772-2550

NEW HAVEN WATER COMPANY
NEW HAVEN, CONNECTICUT

REPORT ON
FLOOD FLOWS AND SPILLWAY CAPACITIES
WEST RIVER SYSTEM DAMS

JANUARY 1967

MALCOLM PIRNIE ENGINEERS
Office Park
226 Westchester Avenue
White Plains, New York 10604

I. PURPOSE AND SCOPE

On June 26, 1965, Mr. Joseph W. Cone, Dam Consultant to the Water Resources Commission, reported to the Commission the results of an assignment by the Commission to study the present condition of the dams owned by the New Haven Water Company on the West River and its tributaries. Mr. Cone's report, which will be summarized later, was not intended to be a comprehensive study of the dams in question. It indicated that spillway capacities on four of the five dams concerned were less than considered desirable, and recommended that a more detailed engineering study be made by the Company to determine deficiencies, if any, and the necessary corrective measures.

Subsequently, Malcolm Pirnie Engineers was authorized to study the adequacy of all spillways in the West River system and make recommendations as to changes and additions.

II. DAMS INVESTIGATED

The dams under investigation store water for the West River or Woodbridge system and are located on the West and Sargent Rivers of Connecticut. The dams impound runoff from a total drainage area of 13.6 square miles, the southern extremity of which lies approximately one and four-tenths miles north of the New Haven city line. The system has a yield of about 10 million gallons per day.

The following tabulation contains pertinent data concerning the dams and reservoirs studied.

	Bethany	Watrous	Chamberlain	Glen	Dawson
Date Built	1892-1931	1914	1899-1959	1907	1889
Drainage Area S.M.					
Direct*	3.8	3.3	4.0	1.7	0.8
Total	3.8	7.1	4.0	5.7	13.6
Res. Cap. MG	650	725	894	197	325
Res. Area, Acres	105	109	102	26	69.5
Spillway Data					
Elev., MSL	431 432	223 ~~224~~	398	213 220	161 ~~157.5~~
Freeboard, Ft.	4.25	5.0	12.0	4.0	6.0
Length, Ft.	80	70	50	40	80

*Does not include drainage area above upstream dam.

Additional data are as follows:

Bethany - Gravity masonry section built in 1892, faced with concrete in 1931. Downstream embankment. Spillway on dam

crossed by bridge of limited headroom. Downstream channel not limiting.

Watrous - Lies two miles downstream from Bethany Dam on West River. Watrous is a gravity concrete section with an earth embankment on the downstream side. Its spillway is not obstructed and the channel leading from the spillway is not limiting. Watrous Dam is about 0.6 miles upstream from Lake Dawson.

Chamberlain - Chamberlain was built of earth on the Sargent River branch of the West River, with a masonry core wall, in 1891. It was raised 35 feet and a new spillway was constructed in 1958-59. It has a side channel spillway with ample downstream channel capacity.

Glen - Glen Dam is a gravity concrete structure on the Sargent River one and one-half miles below Chamberlain Dam.

Dawson - Dawson Dam was built in 1889. It is an earth structure with a concrete core wall. The spillway channel was damaged in the 1955 hurricane flood and rebuilt shortly thereafter.

The West River continues to flow in a southerly direction below Lake Dawson, passing through Konolds Pond and between New Haven and West Haven to Long Island Sound, about six miles away.

III. REPORT OF STATE WATER RESOURCES COMMISSION

Mr. Joseph Cone's report considered flood experiences at each of the West River dams and estimated the flows that spillways of these dams could carry safely. The report did not include a detailed study and was in effect a reconnaissance study of the structures in question. A detailed study was left up to the Company, and this present report concerns more detailed studies of each dam and spillway.

Mr. Cone's conclusions are summarized as follows:

(1) A storm with a recurrence interval of 1,000 years probably should be used in studying dam safety.

(2) The most severe storm of record in the West River area, that of October 1955, was probably one with a recurrence interval of less than 100 years.

(3) The West River drainage area is approximately at the lower size limit of the Connecticut Formula. Flood flow from its smaller parts can probably be better estimated using the formula below:

$$Q = RF \times LF \times FF \times 9A^{2/3}$$

Q = Flow, cfs
RF = Rainfall Factor
LF = Ground Cover Factor
FF = Frequency Factor
A = Area in Acres

(4) Spillway capacities of the five reservoirs of the West River system are estimated as follows:

Dam	cfs	csm
Bethany	1,980	540
Watrous	2,660	380
Chamberlain	6,300	1,525
Glen	1,120	195
Dawson	2,870	215

(5) The report concludes as follows:

(a) Bethany should be able to carry a flow of over 4,000 cfs and with a 1,000-year storm would be overtopped by one foot.

(b) Watrous spillway will barely carry flood from its direct watershed and hence is deficient in capacity by the flow from Bethany or 4,000 cfs.

(c) Chamberlain has an adequate spillway.

(d) Glen was nearly overtopped in 1955 and will be overtopped by a greater storm.

(e) Dawson was nearly overtopped in 1955 and can be expected to be overtopped with any greater storm.

(6) It recommends a comprehensive study with corrective measures to be applied as soon as possible.

These estimates indicate that at peak flow the Bethany Reservoir is about 1.3 feet below the top of the dam; Chamberlain Reservoir is about 7.7 feet below the top of the dam; and Watrous Reservoir is about 0.3 feet below. Both Watrous and Bethany are masonry sections and little or no freeboard is essential, although some is usually allowed to prevent waves from splashing over the dam.

The spillway at Glen Dam will presently carry about 1,200 cfs before the dam is overtopped. It is estimated that this storm is of the magnitude that has a recurrence interval of about 300 years. The 1,000-year storm, as used in this report, would produce a reservoir elevation about 1.0 foot above the top of the dam. The dam is of masonry and could withstand overtopping. The overflow would be voluminous and would result in considerable erosion below the dam. In our opinion the risk is too great to continue operation of this reservoir with the present spillway capacity even though overtopping of this reservoir is not likely to cause danger to life and the property of others below the West River system. Methods of increasing spillway capacity are discussed in Section VI.

Dawson spillway will carry a flood of 3,620 cfs with no freeboard. With 2 feet of freeboard, the minimum we consider feasible for this dam, the spillway will carry about 1,900 cfs. The estimated outflow for the 1,000-year storm is 5,300 cfs. In our opinion the Dawson spillway can safely carry a storm with a frequency of about 150 years. Dawson is the lowest

dam in the series on the West River system and is located above a populated and developed area that probably would suffer severe damage and possible danger to life in case of failure. As it is an earth dam that must not be overtopped, even by wave runup, its spillway capacity must be increased materially. Methods of doing so are discussed in Section VI.

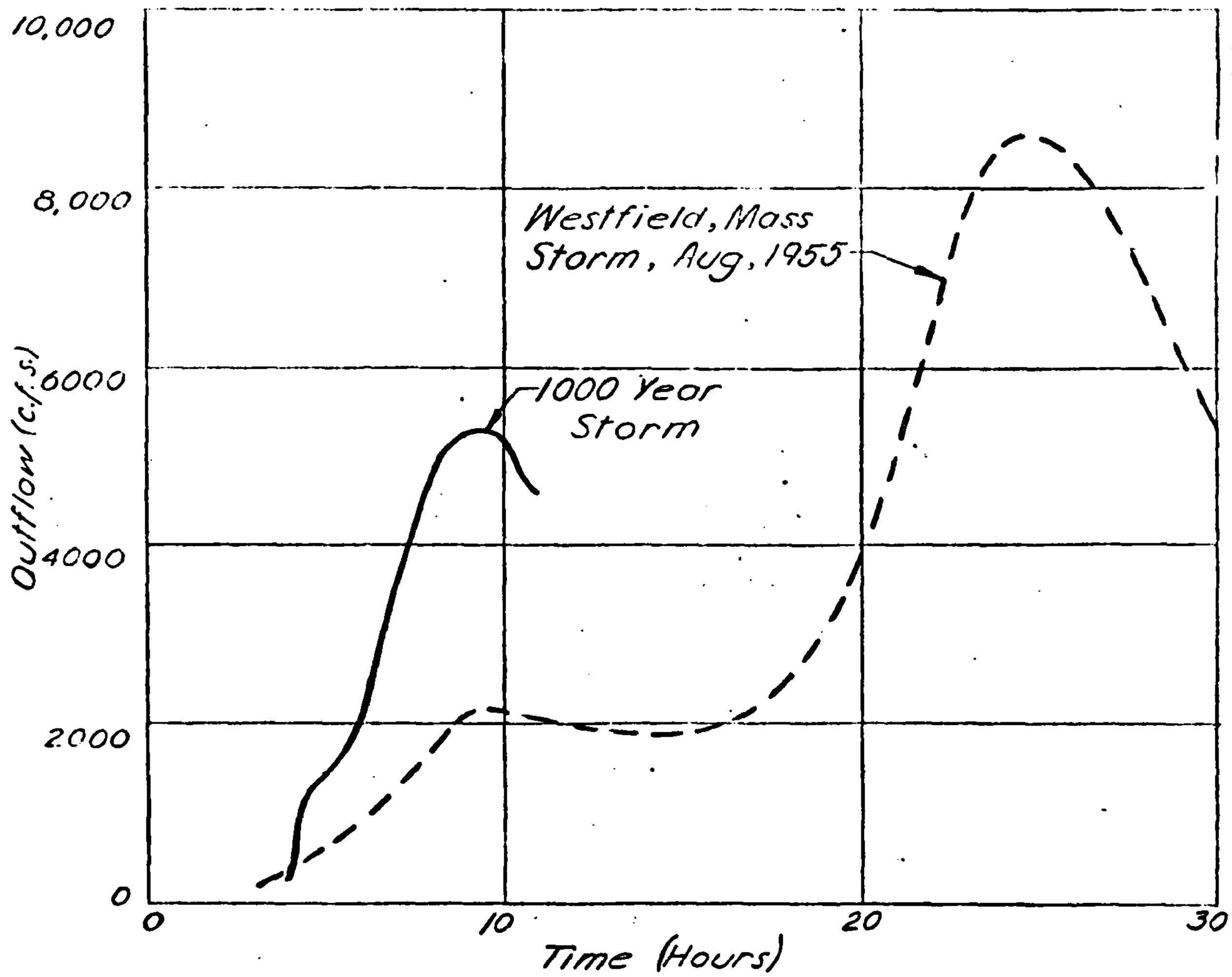
Westfield, Massachusetts, Storm of 1955

To investigate the effect of a storm similar to the Westfield, Massachusetts, storm of August 1955, the Norfolk, Connecticut, recording rain gage record of the storm was adjusted to equal the 24-hour readings taken at the Westfield gage and the resulting storm was transposed to the West River watershed. Hydrographs were constructed for runoff from the storm, which flows were routed through the reservoir system.

This storm produced much more water than the 1,000-year storm, and the peak flows are of the magnitude of 50 per cent greater. The following tabulation compares the two storms.

Reservoir	Outflow from Reservoir, cfs: 1,000-Year Storm	Outflow from Reservoir, cfs: Westfield Storm	
Bethany	1,500	2,200	1980
Chamberlain	1,800	2,700	6300
Watrous	2,800	4,300	2660
Glen	2,300	3,800	1200
Dawson	5,300	1'FB 8,700	3620

Figure 2 shows a visual comparison of the two storms in terms of outflow from Dawson Reservoir.



DAWSON

COMPARISON OF OUTFLOW HYDROGRAPHS - 1000 YEAR STORM VS. WESTFIELD, MASS. 1955 STORM

The Westfield storm produces outflows within the spillway capacities of Bethany wrong and Chamberlain. Watrous is overtopped by about 0.2 feet. In view of the uncertainty of the estimates and the construction of the dam, this slight overtopping does not appear of great concern.

Both Glen and Dawson would be overtopped to a greater extent than in the 1,000-year storm, and this factor has been kept in mind in considering methods of increasing spillway capacity discussed in Section VI.

VI. METHODS OF INCREASING SPILLWAY CAPACITY

In our opinion there is no need to consider modification of the Bethany, Chamberlain or Watrous spillways to provide additional capacity to carry flood flows. Serious consideration must be given to the effect of probable future flood flows at Glen and Dawson spillways.

Glen Dam

For Glen spillway to carry the 1,000-year flood without overtopping the present dam and without use of the blowoff will require increasing the spillway length to 78 feet or, with present length, increasing the freeboard to 6.0 feet. (Spillway ... Elev. 213.8) To carry the Westfield storm requires increasing the spillway length to 95 feet or the freeboard to 9.2 feet using the present length. (Spillway 215.6) The factor of safety against overturning for Glen Dam, as determined in Section VIII, is as small as can be tolerated when the water level is 4 feet above the spillway crest, so raising the dam does not appear feasible.

It appears possible to add the required length by building an extension to the existing spillway at a 90 degree angle or by installing an auxiliary spillway at the north end of the dam. Either is feasible, although there are advantages to confining such work to the present spillway location so a common discharge channel may be used. The existing spillway may also be replaced by a side channel spillway 95 feet long.

Glen Reservoir has a small storage capacity and is principally used to provide workable head conditions for

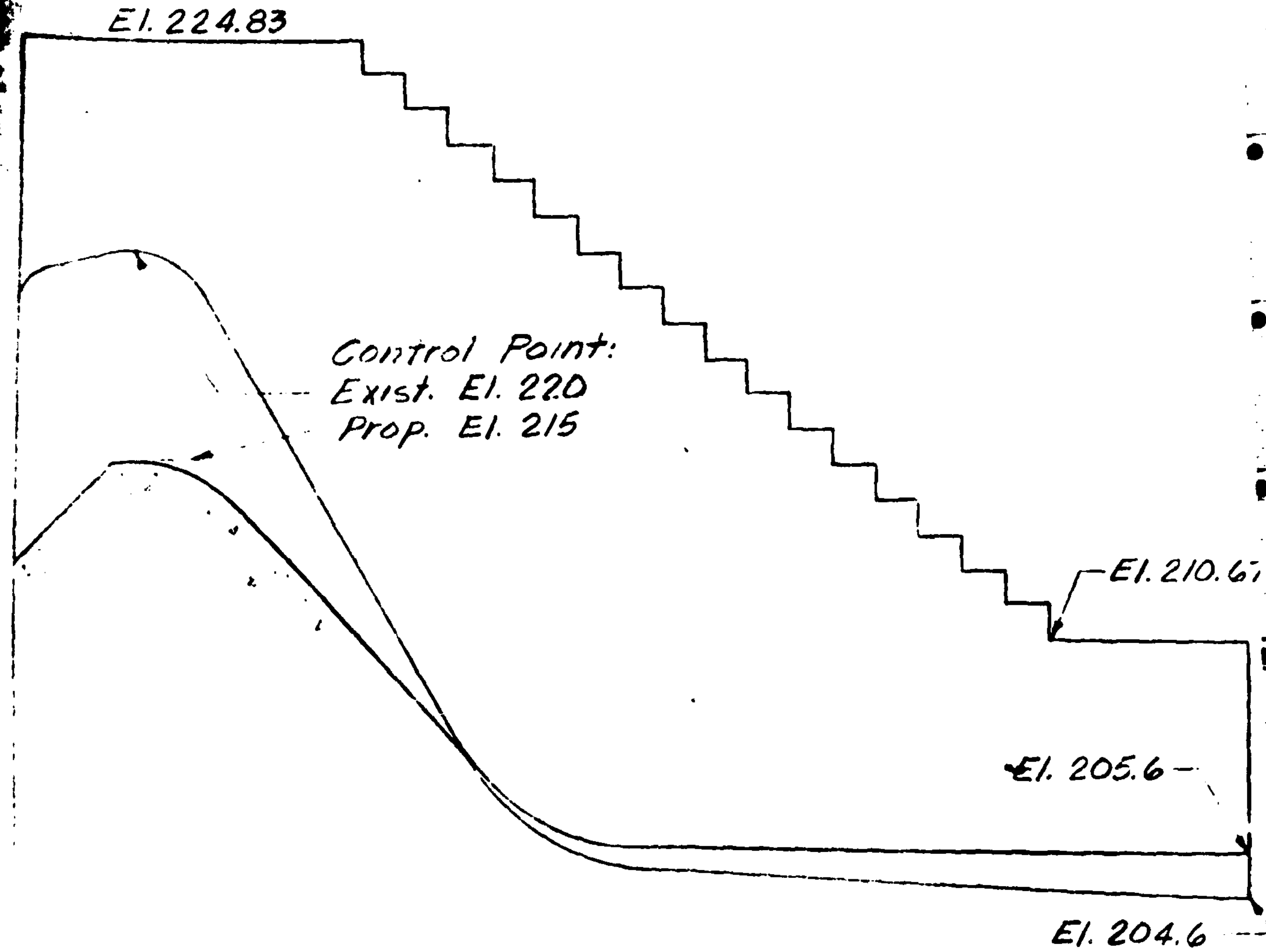
the Sargent River portion of the West River system. Lowering the normal reservoir level 5 feet would decrease storage by about 40 million gallons and would reduce the yield of the West River system a very small amount. At full reservoir, pressures would be reduced about 2 psi.

Although the urgency of providing more outflow capacity for Glen is not as great as for Dawson, it is advisable to modify the Glen spillway at an early date. The least expensive method of doing so appears to be by lowering the spillway crest as shown in Figure 3 to permit passing future extreme floods without overtopping the non-overflow section of the dam. For the 1,000-year storm used in our study, the spillway should be lowered at least 2 feet. For the Westfield storm it should be lowered 5 feet. In view of the uncertainty of all methods of estimating future flood conditions and the minor effect on system operation if this plan is followed, we recommend lowering the Glen spillway by 5 feet. The cost of cutting down and reshaping the spillway crest is estimated to be of the magnitude of $5,000. The work does not require extensive preparation and can be started at any time.

Crest gates could be installed on the spillway after lowering to maintain present storage. They would add approximately $100,000 to the cost.

An alternate method of obtaining the necessary spillway capacity while maintaining present water levels would be to rebuild the spillway. This alternate will cost about $100,000, approximately the same as the crest gate alternate. Given the

Fig. 3



SECTION THROUGH SPILLWAY
SCALE: 1/4" = 1'-0"

PROPOSED METHOD OF LOWERING GLEN SPILLWAY

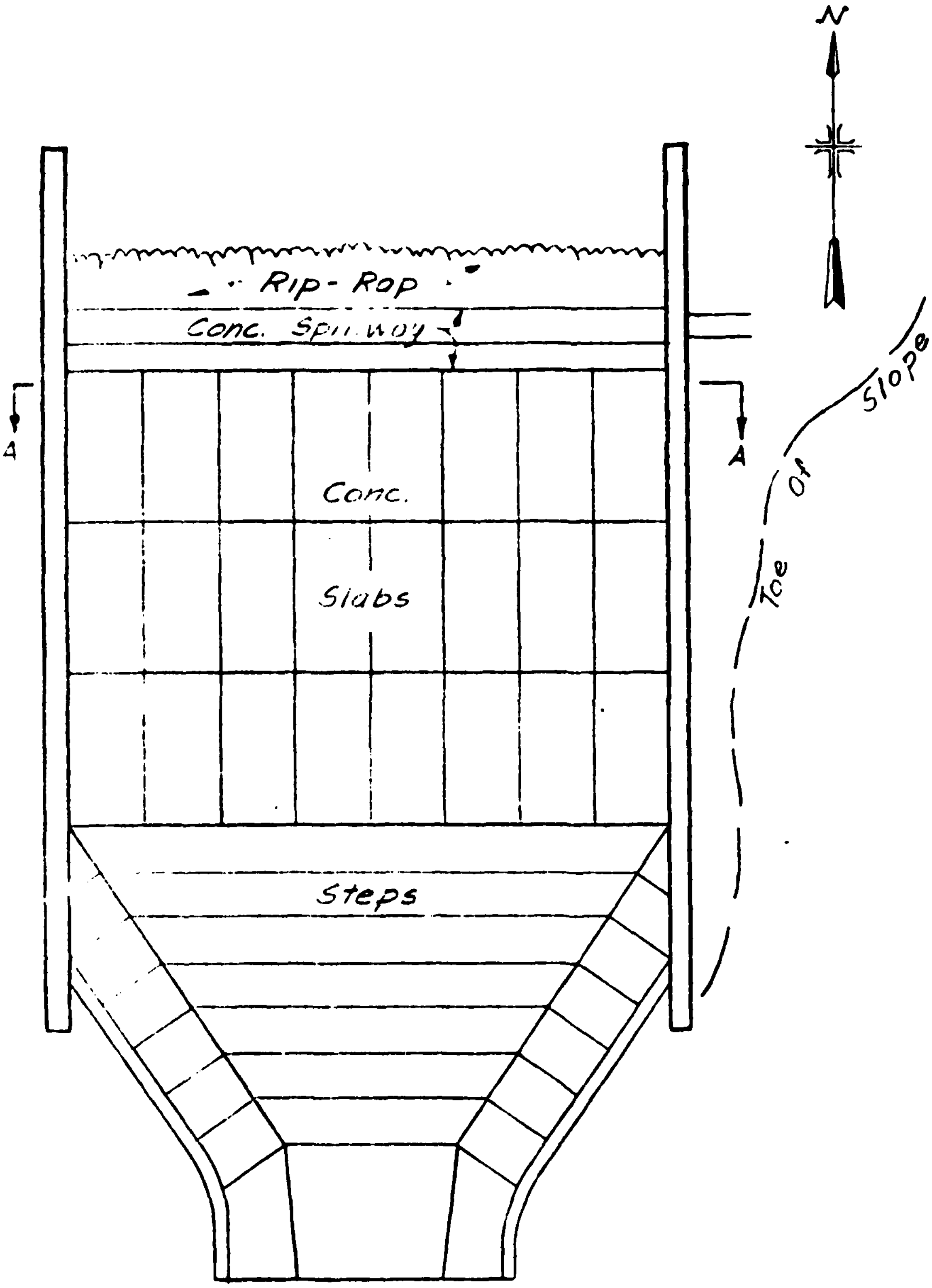
B-31

two choices, we prefer extending the fixed spillway rather than utilizing crest gates with their attendant maintenance and operation problems.

Dawson Dam

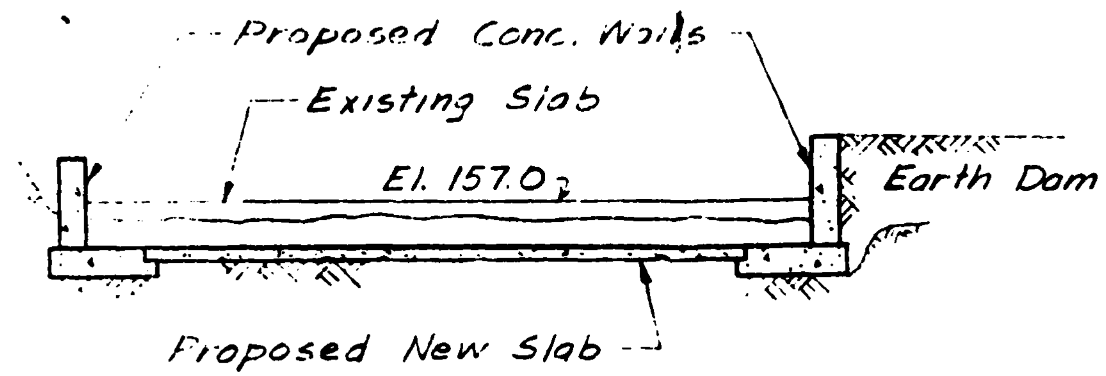
The spillway at Dawson Dam now is 80 feet long. To carry a flood of 5,300 cfs, without freeboard, the spillway must be lengthened by about 40 feet. With 2 feet of freeboard, the length must be increased 140 feet. To carry the Westfield storm of 8,700 cfs without freeboard would require extending the spillway 115 feet. Extensions beyond about 30 feet by projection of the spillway line are difficult because of topographical conditions. Extending a side channel spillway northward alongside of the reservoir would necessitate channel construction through the existing spillway channel. Detailed studies have not been made, but preliminary examination indicates that it will be less costly to lower the existing spillway. If the spillway is lowered 5 feet and 2 feet of freeboard are allowed, it will carry a flow of about 7,200 cfs. This is more than the 1,000-year flood of 5,300 cfs and less than the Westfield storm of 8,700. The Westfield storm would reduce freeboard to about 1 foot. The cost of lowering the spillway 5 feet would be somewhat greater than lowering it 3 feet, which would allow no freeboard for the 1,000-year storm, but the major difference would be in rock excavation, and the added safety would be worth the difference in cost. We recommend lowering the spillway 5 feet as shown in Figure 4, at an estimated cost of $125,000.

Fig 4

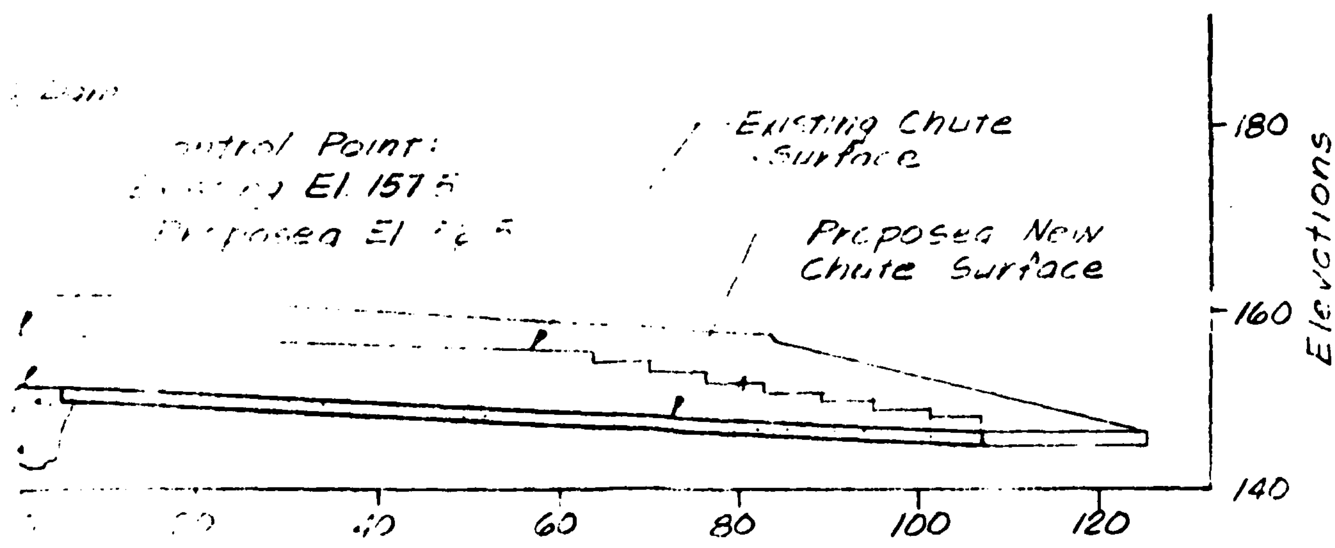


PLAN

Fig. 4



SECTION AA



LONG. SECTION

PROPOSED METHOD OF
LOWERING DAWSON SPILLWAY
SCALE 1" = 20' HORIZ & VERT.

Dawson Reservoir is at too low an elevation for direct service, and its yield is now pumped into the system when needed. Lowering the spillway 5 feet would reduce storage by about 110 million gallons and would reduce slightly the yield of the West River system. Other considerations may indicate the need of maintaining water levels at present flow line elevation. If so, crest gates may be installed at a cost of approximately $150,000, making the total cost of the work approximately $275,000.

VII. EFFECT ON YIELD OF LOWERING SPILLWAY CRESTS

If Glen spillway is lowered 5 feet, storage is reduced about 40 mg. If Dawson is lowered 5 feet, the loss in storage is 102 mg. Total storage loss if both spillways are lowered is 142 mg.

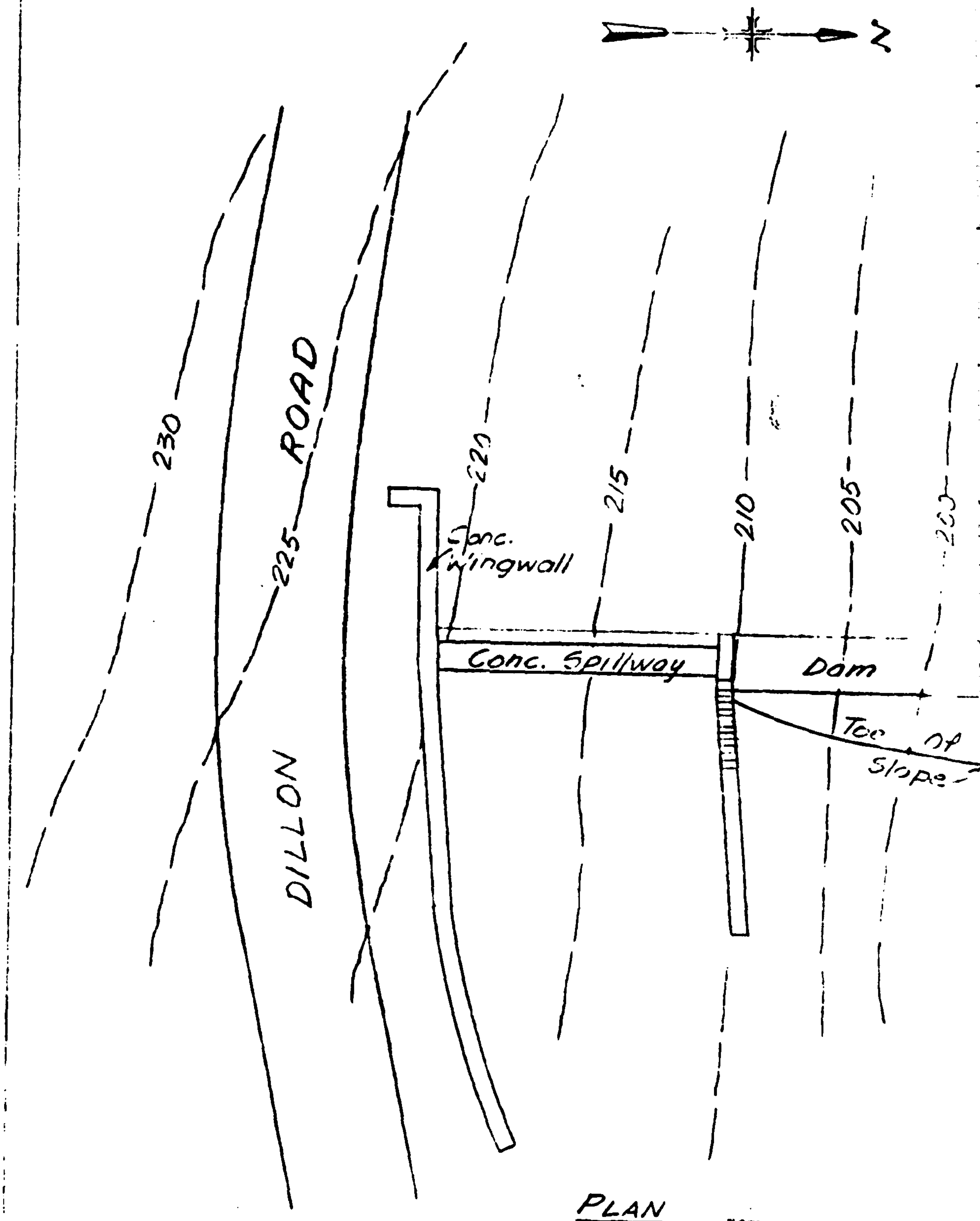
During the 1964-66 dry period, water produced from May 20, 1964, to November 1, 1966, averaged about 8.1 mgd. The minimum amount left in storage in February 1966 was about 626 mg. With no reserve allowance, and assuming that the reservoirs refill by next June, the supply could have been increased about 0.6 mgd and the system yield would be 8.7 mgd. If 20 per cent storage was allowed for emergency reserve, the yield would be approximately 8.2 mgd.

The loss in storage by lowering the spillway would have decreased yield over this dry period by 0.13 mgd. During wet periods when the system refills each year, loss of yield would be greater and, in a year when Dawson is below flow line level for a 6-month period, the reduction would be about 0.8 mgd.

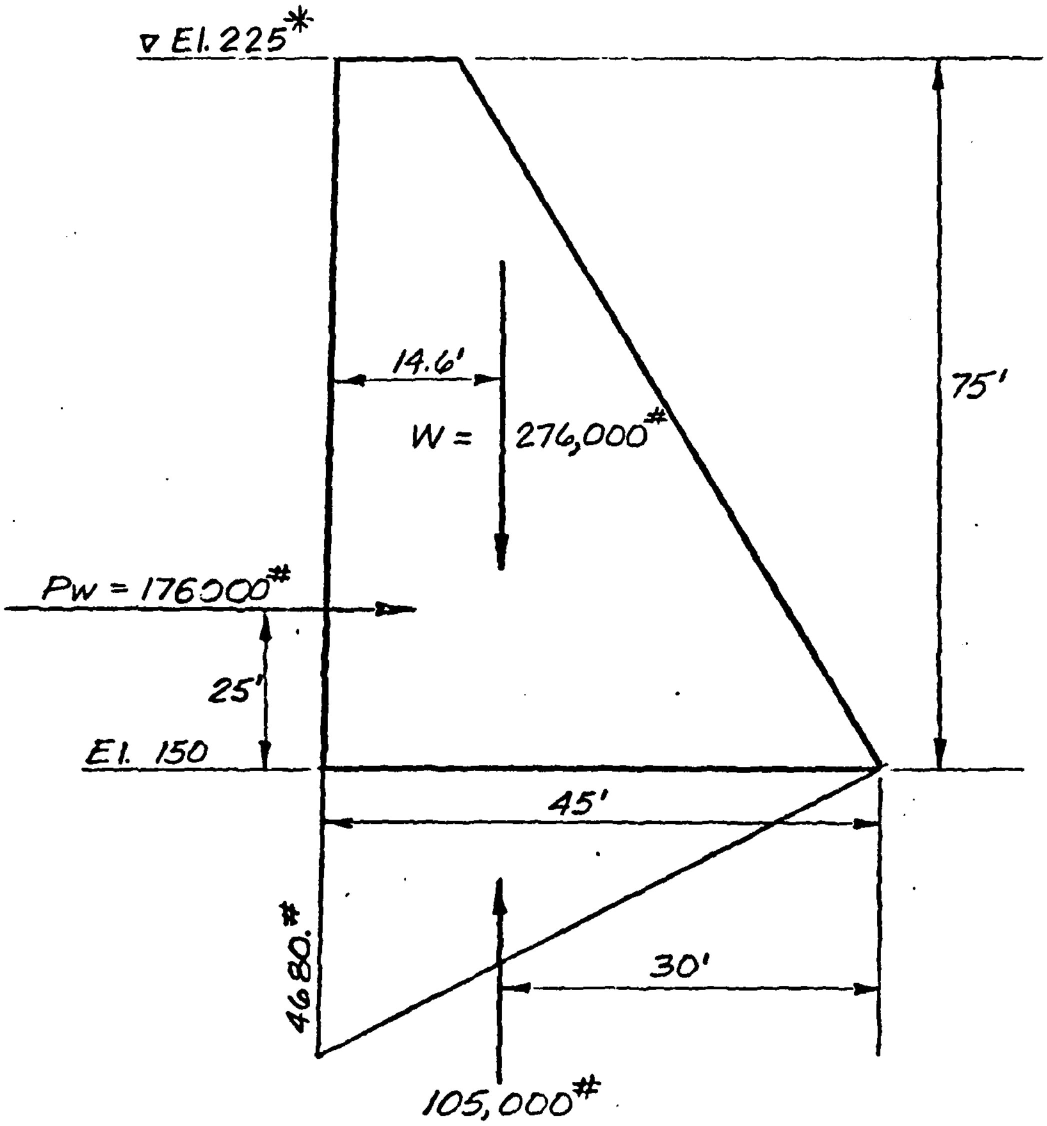
VIII. STABILITY OF DAMS

Stability of each dam in the West River system has been investigated. Chamberlain and Dawson are earth dams with satisfactory sections. Bethany and Watrous are masonry dams with massive earth backup on the downstream side. There is no question as to their stability.

Glen is an exposed masonry dam and its stability has been investigated against overturning. Because of the construction, it is safe against sliding. When full to the crest of the non-overflow section, presently 4 feet above the spillway elevation, the factor of safety against overturning is 1.18. If the dam is raised one (1) foot, which could be easily done, the factor of safety decreases to 1.11, as shown in Figure 5. Since uplift is probably less than assumed, we estimate that the dam is safe against overturning as long as the maximum water level does not exceed the top elevation of the existing non-overflow section. We do not recommend any increase in height.



PLAN
SCALE: 1" = 20'



$$\text{F.S. - OVERTURNING} = \frac{276 \times 30.4}{(176 \times 25) + (105 \times 30)} = 1.11$$

(With top El. 225 - assuming dam raised one(1) foot.)

STABILITY ANALYSIS – GLEN DAM

*Present Elevation 224

IX. RECOMMENDATIONS

We recommend that the Water Company increase the capacity of the spillways at Glen and Dawson Dams by lowering each of these spillways approximately five (5) feet. Since a major storm may occur at any time, the work should be done as soon as possible.

NEW HAVEN, CONNECTICUT

COMMISSION
RECEIVED
NOV 9 1967
ANSWERED
REFERRED
FILED

MEMORANDUM REPORT TO WATER COMPANY
ON
INVESTIGATION OF THE EFFECTS OF A FLOOD
PRODUCED BY THE MAXIMUM POSSIBLE STORM
ON SPILLWAYS OF WEST RIVER SYSTEM

AUGUST 2, 1967

The effect of the "maximum possible storm" on the West River System is reported in this memorandum.

The "maximum possible storm" employed is defined and quantitatively estimated in U. S. Weather Bureau Hydrometeorological Report No. 33 entitled "Seasonal Variation of the Probable Maximum Precipitation East of the 105th Meridian for Areas from 10 to 1,000 Square Miles and Durations of 6, 12, 24 and 48 Hours." The report defines the "maximum possible precipitation" as "the critical depth-duration-area rainfall relation for a particular area during various months of the year that would result if conditions during an actual storm in the region were increased to represent the most critical meteorological conditions that are considered probable of occurrence."

As shown on Exhibit 1, the rainfall totals used for the West River System analyses are for durations of 6 and 12 hours on an area of 10 square miles for September -- the most severe month for the vicinity of New Haven, Connecticut. The hourly

distribution of the total rainfall assumed is according to Figure 4, page 32 of U. S. Department of the Interior publication "Design of Small Dams." The distribution is a comparatively severe one with 50 per cent of the 6 hour total falling within 1 hour.

The sequence in which the hourly totals were arranged is in accordance with the recommendation made on page 50 in "Design of Small Dams." The arrangement of the 12 hourly increments is 11, 9, 7, 5, 3, 1, 2, 4, 6, 8, 10, 12, where the number represents the order of magnitude with the lowest number representing the largest magnitude. This arrangement gives a flood greater than one based on the assumption that the greatest hourly increment of rain occurs during the first hour of a storm

The effective, runoff-producing rainfall was estimated by subtracting 1 inch initial infiltration and 0.1 inch per hour thereafter from the total rainfall.

In order to pass the unusually high flows for the "maximum possible storm," several modifications of both the length and crest height of spillways were tried. Spillway rating curves and stage capacity curves for each of the five reservoirs are shown on Exhibit 2 and Exhibit 3, respectively.

The unit-hydrographs and routing procedures employed are those outlined in our report of January, 1967. Detailed computations are shown on Exhibit 4, pages 1 through 8.

The inflow-outflow curves for each of the reservoirs are shown on Exhibit 5, pages 1 though 3. As no significant storage effect is obtained from Lake Dawson, the outflow

hydrograph as shown on Exhibit 5, page 5, will be the same with a spillway 250 feet long.

The "maximum possible" flood outflows at each of the West River reservoirs and the conditions at the Spillways are summarized below:

Dam	Peak Spillway Discharge cfs	Free-Board ft.	Maximum Head (ft.)	
			Over Spillway	Over Dam Crest
Chamberlain	7200	12.0	10.8	-1.2
Glen	9665	9.0*	11.3	+2.3
Bethany	7350	4.25	5.2	+1.0
Watrous	15,400	5.0	7.1	+2.1
Dawson				
80' Spillway	26,260	11.5*	13.8	+2.3
250' Spillway	26,260	11.0*	9.0	-2.0

*Freeboard above proposed new sill elevation

NEW HAVEN WATER COMPANY
NEW HAVEN, CONNECTICUT

DESIGN REPORT

DAWSON SPILLWAY

Our report of January 1967 recommended lowering the spillway of Dawson Dam five feet to increase its capacity to approximately 7,200 cfs. This report was reviewed by the Water Resources Commission of Connecticut which requested that an investigation be made of the effect of routing a maximum possible flood through the West River System. This study was made.

The following tabulation shows the estimated floods that would occur under varying conditions.

Estimate of Record	-	2,300 cfs
1000 year storm	-	5,300 cfs
Transposed Westfield Storm	-	8,700 cfs
Maximum Possible	-	27,000 cfs

Conditions at the dam and downstream from the dam were considered and it was concluded that any storm materially exceeding the 1000 year storm would cause great damage downstream and that the damage would not be materially greater if Dawson Dam were breached during the storm. After reviews with the Water Company it was decided to submit plans to the Water Resources Commission that would pass the 1000 year storm and such additional flow as could be carried by widening the spillway a reasonable amount.

We have made studies of alternate designs of the channel downstream from the spillway. Our January 1967 report showed a chute type spillway with a slope of about 4 per cent. The chute must be curved in plan to suit the topography. At the higher flows investigated unstable flow conditions around the curve result in very high water elevations at the outer edge of the curve with transitory waves which create in our opinion a dangerous hydraulic condition. To be sure of containing the waves would require very high chute walls and a very elaborate energy dissipation device at the end of the chute would be required.

Our studies indicate that a very much safer design will result with less than critical slopes in the outlet channel and that the cost will be less.

The stepped channel design which results from the use of flatter slopes can be built in various widths. The narrowest feasible width is found to be 50 feet. At this width the critical depth which will occur at each drop in the channel floor is approximately 7 feet for the 1000 year storm of 5300 cfs.

The present spillway is 80 feet wide. We find it feasible to increase this width to 110 feet. The required head on an ogee spillway for the 1000 year storm is 5.5 feet. Using this as a design storm and a freeboard of 3.5 feet the spillway crest should be 9 feet below the top of the dam or at Elevation 155.0.

At the above crest elevation and no freeboard the spillway will carry 10,400 cfs which is more than the 1000 year or Westfield transposed storms. At this flow critical depth in the 50 foot wide channel is about 11 feet.

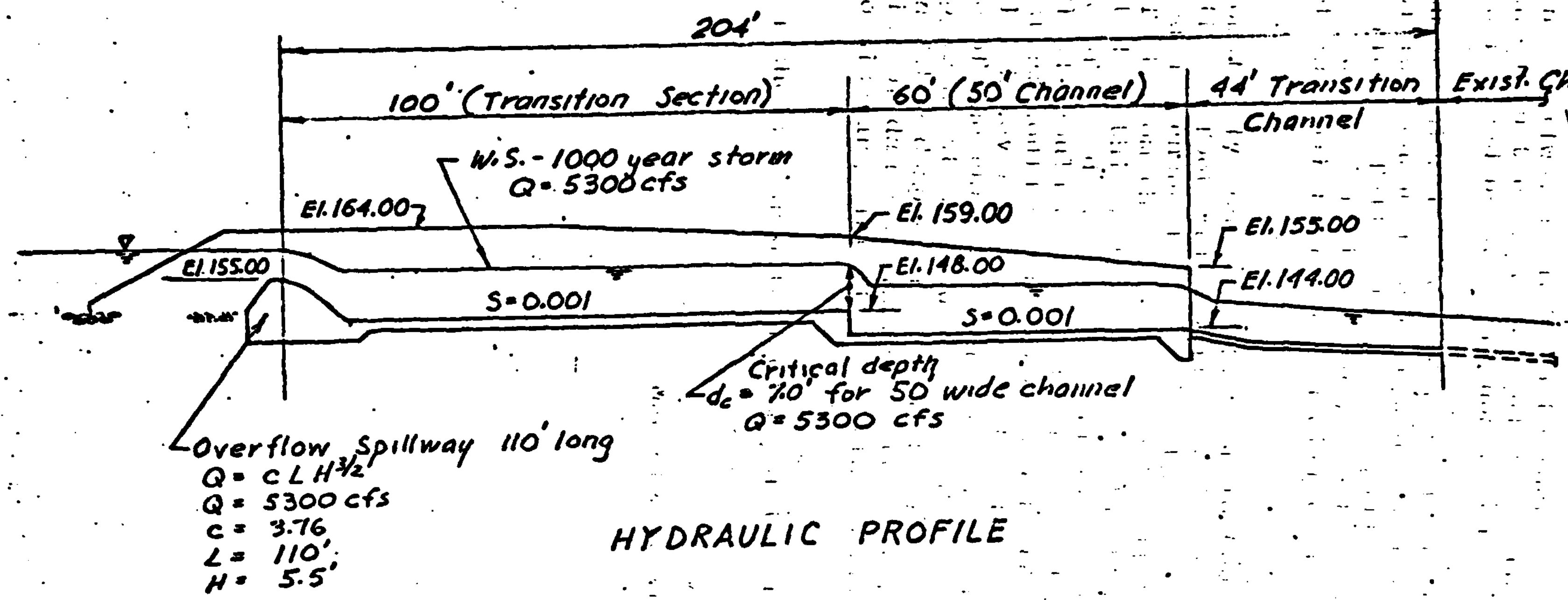
If the spillway is lowered to increase flow the downstream channel must be widened, otherwise the flow over the spillway is reduced because of submersion.

In our opinion the most reasonable design is for a stepped spillway channel 50 feet wide with a capacity of 10,400 cfs flowing full and an ogee spillway 110 feet long. The spillway will carry the 1000 year storm with a freeboard of 3.5 feet and with no freeboard will carry a storm of 10,400 cfs.

To completely contain a flow of 10,400 cfs would require extending the 50 foot wide spillway channel about 340 feet. The proposed new channel intersects the existing paved channel about 204 feet from the overflow crest. If the new channel is terminated at this point, and the old channel used for the balance of the distance, flood flows up to approximately historic floods could be carried. Larger floods including the 1000 year storm would not be contained in the old portion of the channel. However no damage to the dam could occur and the convex side of the curved channel is on the uphill side with bed rock apparent at shallow depth. All erosion that could occur is on Water Company land and the Water Company prefers to take the risk of damage. Therefore the spillway channel will be built to discharge into the existing channel as shown on the plans.

Fig. 1 shows hydraulic details of the design.

MALCOLM PIRNIE ENGINEERS



HYDRAULIC PROFILE

MALCOLM PIRNIE ENGINEERS
226 WESTCHESTER AVENUE
WHITE PLAINS, NEW YORK

FORM D-4

STATE OF CONNECTICUT
WATER RESOURCES COMMISSION
State Office Building
Hartford, Connecticut

APPLICATION FOR CONSTRUCTION PERMIT FOR DAM

New Haven Water Company Date July 26, 1968

P. O. Address Box 1470

New Haven, Connecticut Tel. No. 203-772-2550

Location of Structure:

Town Woodbridge (Dawson Dam) Shown on USGS Quadrangle New Haven and Vicinity, Con

Name of Stream West River at 1 inches south of Lat. 41^{o}-22'-3

and 3 inches east of Long. 73^{o}-00'-

Directions for reaching site from nearest village or route intersection: (See sketch on reverse side)

See Attached Plans

This is an application for: (New Construction) (Alteration) (Repair) (Removal)
(check one or more of above)

This pond is to be used for: Storage

Dimensions of Pond: width 1,000 ft. length 3,500 ft. area 69.5 acres

Maximum depth of water immediately above dam: 5.0' without temporary flashboards; 7.5' with temporary flashboards

Total length of dam: 900 ft.

Length of spillway: 110 ft.

Height of abutments above spillway: 9 ft.

Type of spillway construction: Concrete

Type of dike construction: Existing earth structure with a concrete core wall

Spillway section will be set on: (Bedrock) (Gravel) (Clay) (Till)
(check one of above)

Remarks: Spillway section will be set on bedrock or compacted dam material including old core wall.

Signed: Joseph A. Novaro, Chief Engr
(owner)

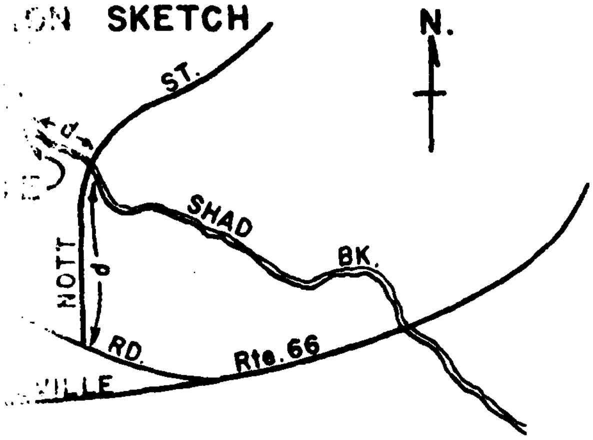
Name of Engineer, if any MALCOLM PIRNIE ENGINEERS

Note: Show details of construction on reverse side

B-47

Show only features of sample which are applicable and dimensions which reflect your i

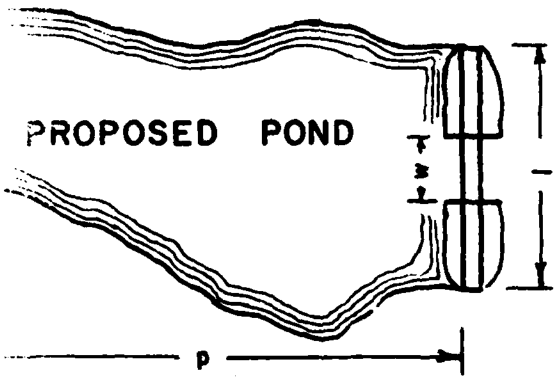
...ON SKETCH



LOCATION SKETCH

See Attached Plans

...LAN



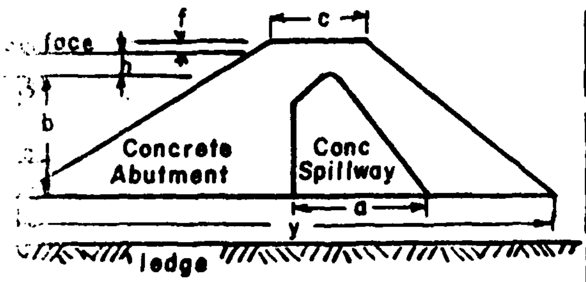
SITE PLAN

l = 900'

p = 3,500'

w = 110'

...Y SECTION

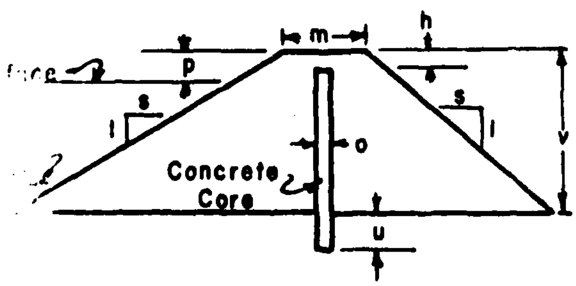


SPILLWAY SECTION

a = 18'
b = varies 10' to 13'
c = 60'
e = 12'
f = 3.5'
h = 5.5' (1,000 year storm)
r = varies 0 to 10'

NOTE...
If there are two methods of discharge Show

...CTION



DIKE SECTION (Existing)

h = 7'
m = 10'
o = 2' min.
p = 6.5' with temporary flashboard
p = 9.0' without temporary flashbo
s = 2:1
t = unknown
u = 5' min. (varies)
v = 50' max. (varies)

August 20, 1968

MEMO TO: File

FROM: William H. O'Brien III

SUBJECT: Lake Dawson Dam - Woodbridge

In reviewing the design for the modifications of the subject dam, the following additional information was obtained from a Mr. Raymond Dugandzic, of Malcolm Pirnie Engineers.

The flashboards are designed to fail with water elevation at 159.5 (two feet above the top of the flashboards). This is based on a yield stress of 35,000 PSI and ultimate strength of 60,000 PSI for the pipe steel.

The proposed spillway will pass the Westfield storm with a freeboard = 1.58 feet.

W.H.O.

INTERDEPARTMENT MESSAGE
STO-201 12-69

SAVE TIME: *Handwritten messages are acceptable. Use carbon if you really need a copy. If typewritten, ignore faint lines.*

TO File	AGENCY Water Resources Commission	DATE March 2, 197?
FROM William H. O'Brien, III Civil Engineer	AGENCY Water Resources Commission	TELEPHONE

SUBJECT Dawson Lake Dam, Woodbridge

On Feb. 24, 1971 the undersigned inspected the subject dam. The work appears to have been well done and in conformance with the approved plans, however, there were several leaks, one of which was quite substantial thru the construction joints of the bottom of the exit channel at the base of the ogee spillway. The water level was approximately 6 inches below the concrete crest of the dam and about 3 feet below the top of the flashboards. These leaks may become substantially greater with a full pond. These leaks however do not appear to effect the safety of the structure.

There was just a slight flow in the new 6 inch outlets through the concrete headwall into the existing stream at the west end of the dam.

W. H. O'Brien III
Civil Engineer

WHO:ljg

March 31, 1971

Mr. Joseph A. Novaro
Chief Engineer
New Haven Water Company
New Haven, Connecticut 06506

Re: Dawson Dam
Woodbridge

Dear Mr. Novaro:

On February 24, 1971 the undersigned inspected the subject dam. There were several leaks, one of which was quite substantial, through the construction joints of the bottom of the exit channel at the base of the ogee spillway. The water level was approximately six inches below the concrete crest of the dam.

It is assumed that the presence of these leaks indicate either a malfunction of the six inch perforated concrete drain pipes under the spillway channel slab or else there is a water barrier preventing water from reaching this perforated pipe. It appears that the possibility exists of a piping condition under the spillway section. We therefore do request information on the as built conditions of the spillway as defined in Section b-b on sheet four of five of the approved plans. The notations indicate that the spillway section may be set on rock in some areas and on compacted dam material in others. We would like a definition of these areas. Thank you for this information.

Very truly yours,

William H. O'Brien, III
Civil Engineer

WHO:ljg

NEW HAVEN Water COMPANY
100 Crown Street / New Haven, Connecticut / 06506 (203) 772-2550

April 7, 1971

State Water Resources Commission
State Office Building
Hartford, Connecticut 06115

Attention: Mr. William H. O'Brien, III
Civil Engineer

Dear Mr. O'Brien:

This is to acknowledge your letter of March 31, 1971 and to confirm my call to you in regard to Dawson Dam.

We inspected the spillway several weeks ago and will make another inspection in warmer weather, at which time, we will write you further.

I have requested AS-built prints from our Consultants and will forward one set to you when I receive them. The entire length of the spillway section was carried down to rock. There were five (5) keyed expansion joints, each being sealed by a rubber expansion seal with the usual circular sections or bulbs on both sides.

Very truly yours,
NEW HAVEN WATER COMPANY

Joseph A. Novaro

Joseph A. Novaro
Vice President-Engineering

JAN:jcp

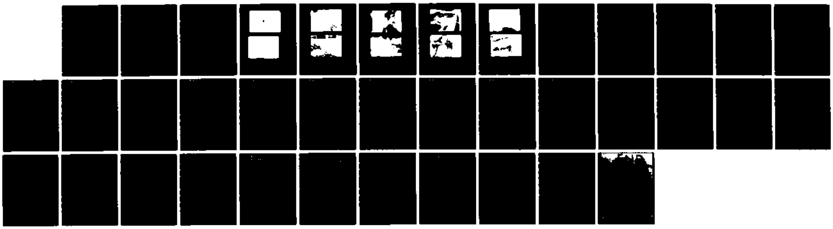
STATE WATER RESOURCES
COMMISSION
RECEIVED
APR 12 1971
ANSWERED
REFERRED
FILED

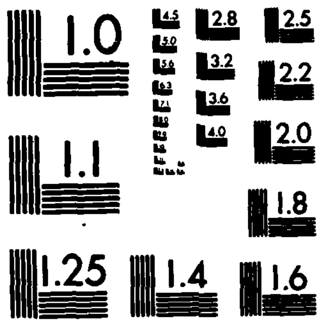
Project Lake Dawson Top of Dam Elevations
Computed By N.N.
Checked By CKG
Field Book Ref.
Other Refs.
Sheet B-2 of
Date 7/5/79
Revisions

stance from utside of right ngwall of pillway	Elevation on ℄ of dam	Distance from outside of right wingwall of Spillway	Elevation on ℄ of dam
0'	164.00	200'	163.9
1'	163.20	250'	163.9
2'	163.36	300'	163.9
3'	163.52	U/S	163.5
4'	163.56	D/S	163.7
5'	163.66	400'	164.1
10'	163.96	450'	163.9
15'	164.3	500'	163.8
20'	164.34	550'	163.8
25'	164.4	600'	163.7
30'	164.5	650'	163.3
35'	164.6	700'	163.5
40'	164.7	750'	163.8
50'	164.6	Bridge	163.86
100'	163.9	Left wingwall elevation	163.9
150'	163.8		

AD-A144 158 NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS 2/2
LAKE DAWSON DAM (CT 0..(U) CORPS OF ENGINEERS WALTHAM
MA NEW ENGLAND DIV AUG 79

UNCLASSIFIED F/G 13/13 NL





MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

APPENDIX C

DETAIL PHOTOGRAPHS

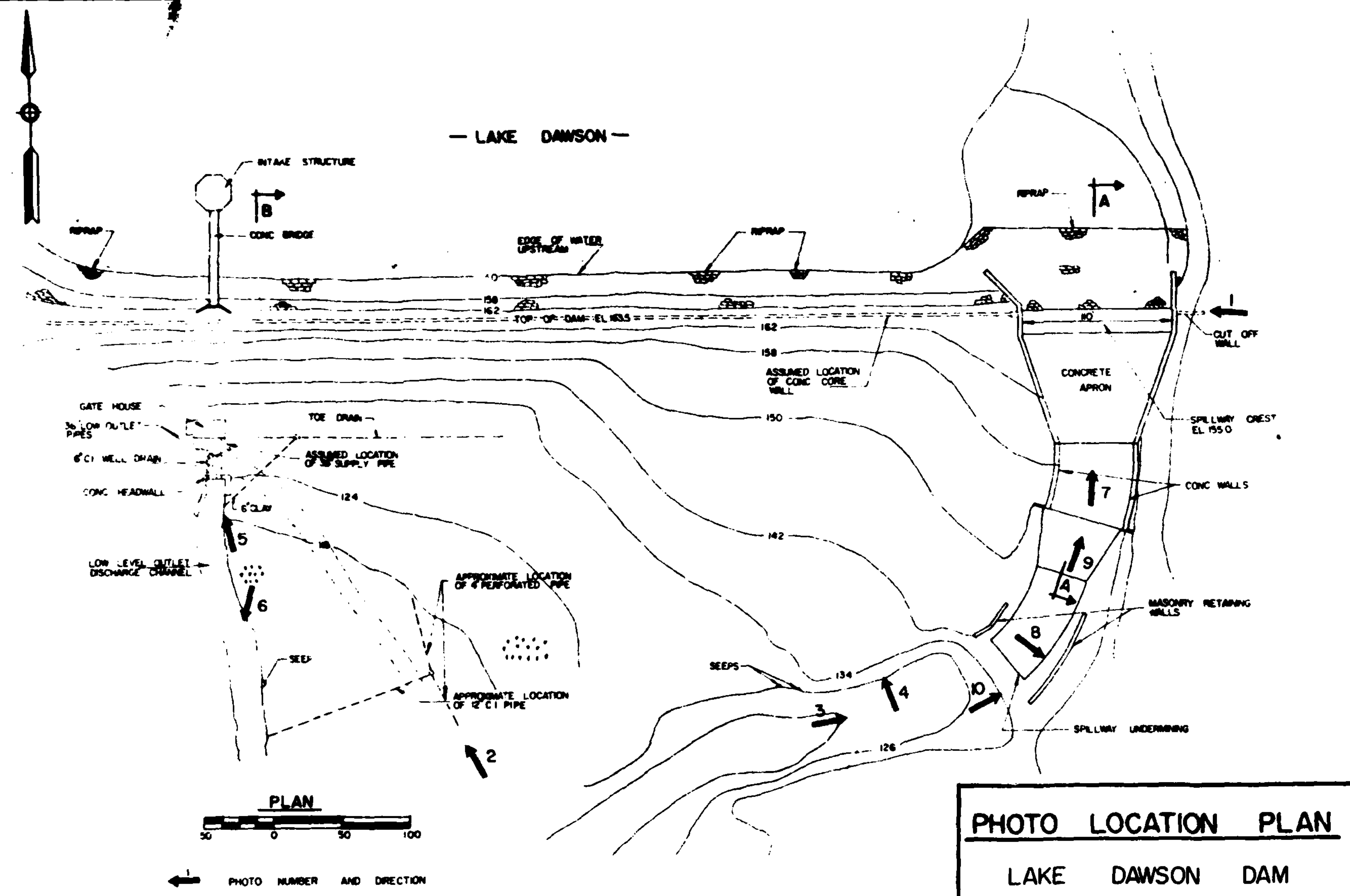


PHOTO LOCATION PLAN

LAKE DAWSON DAM

SHEET C-1

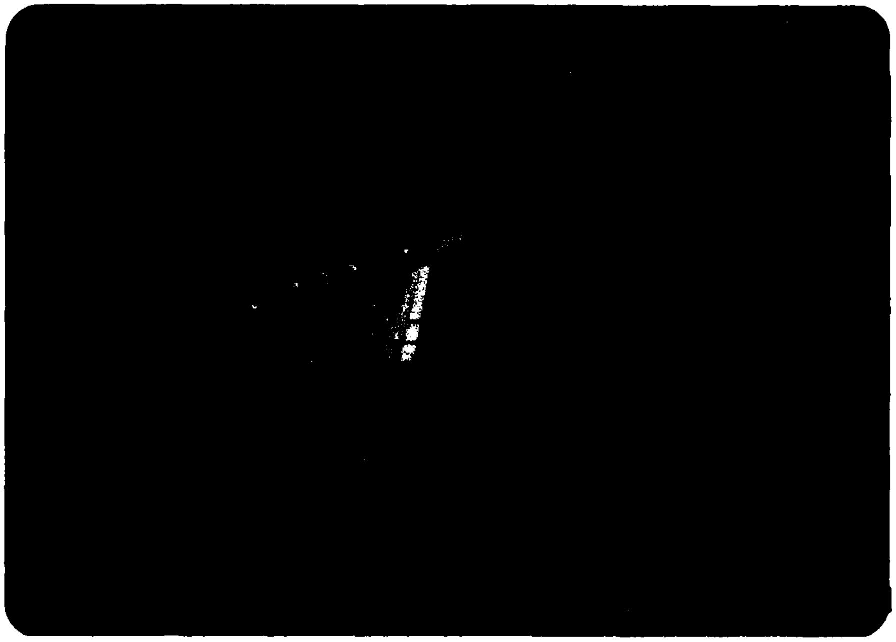


PHOTO 1 - Riprap on upstream slope, spillway and gatehouse as seen from left abutment.

PHOTO 2 - Central portion of downstream slope. Orange circular frame in foreground is the location of the drain pipe outlet from wet area in background, and drain pipe inlet to blowoff discharge channel.

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Lake Dawson Dam West River Woodbridge, Connecticut
CAHN ENGINEERS INC. WALLINGFORD, CONN. ENGINEER		CE# 27 660 KA DATE May '79 PAGE C-1

PHOTO 3 - View of natural spillway channel from downstream.

PHOTO 4 - Seepage on right slope of spillway discharge channel.

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Lake Dawson Dam West River Woodbridge, Connecticut
CAHN ENGINEERS INC. WALLINGFORD, CONN. ENGINEER		CE# 27 660 KA DATE May '79 PAGE C-2

PHOTO 5 - 2-36 inch low level outlets.

PHOTO 6 - Low level outlet discharge channel downstream of dam.
Note minor seepage on the left slope of the channel.

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Lake Dawson Dam West River Woodbridge, Connecticut CE# 27 660 KA DATE May '79 PAGE C-3
CAHN ENGINEERS INC. WALLINGFORD, CONN. ENGINEER		

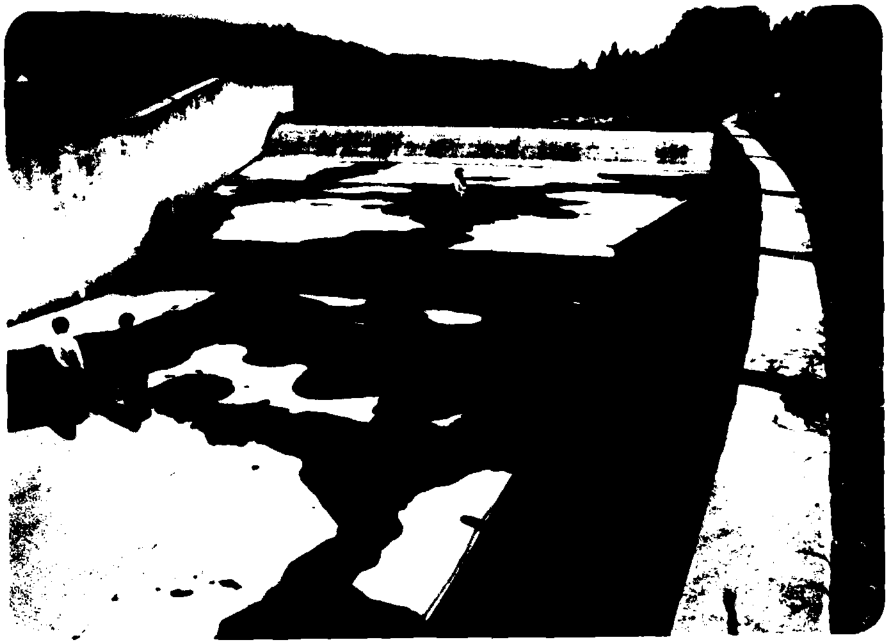


PHOTO 7 - General view of spillway channel and weir from downstream.

PHOTO 8 - Seepage through left stone masonry wall adjacent to concrete spillway channel.

US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Lake Dawson Dam West River Woodbridge, Connecticut
CAHN ENGINEERS INC. WALLINGFORD, CONN. ENGINEER		CE # 27 660 KA DATE May '79 PAGE C-4

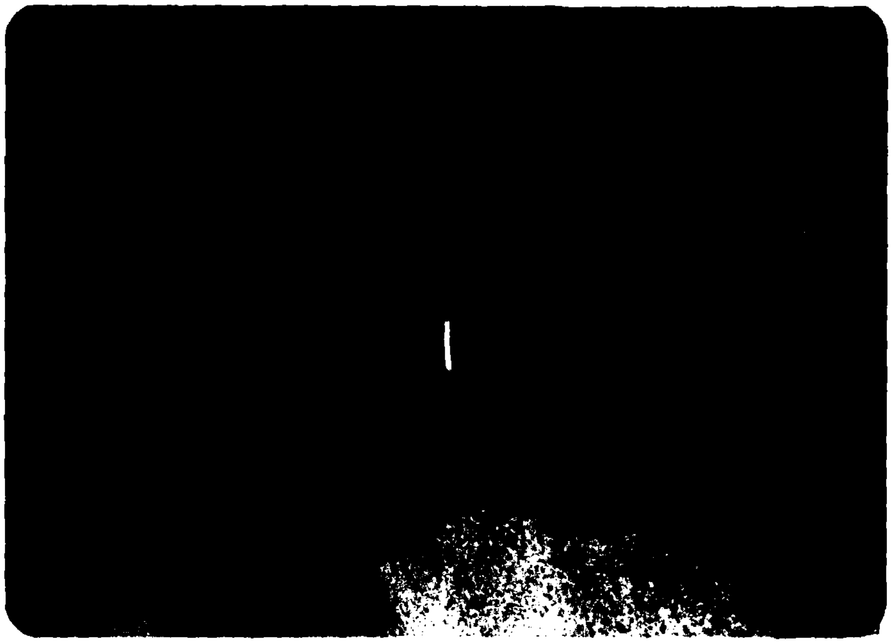


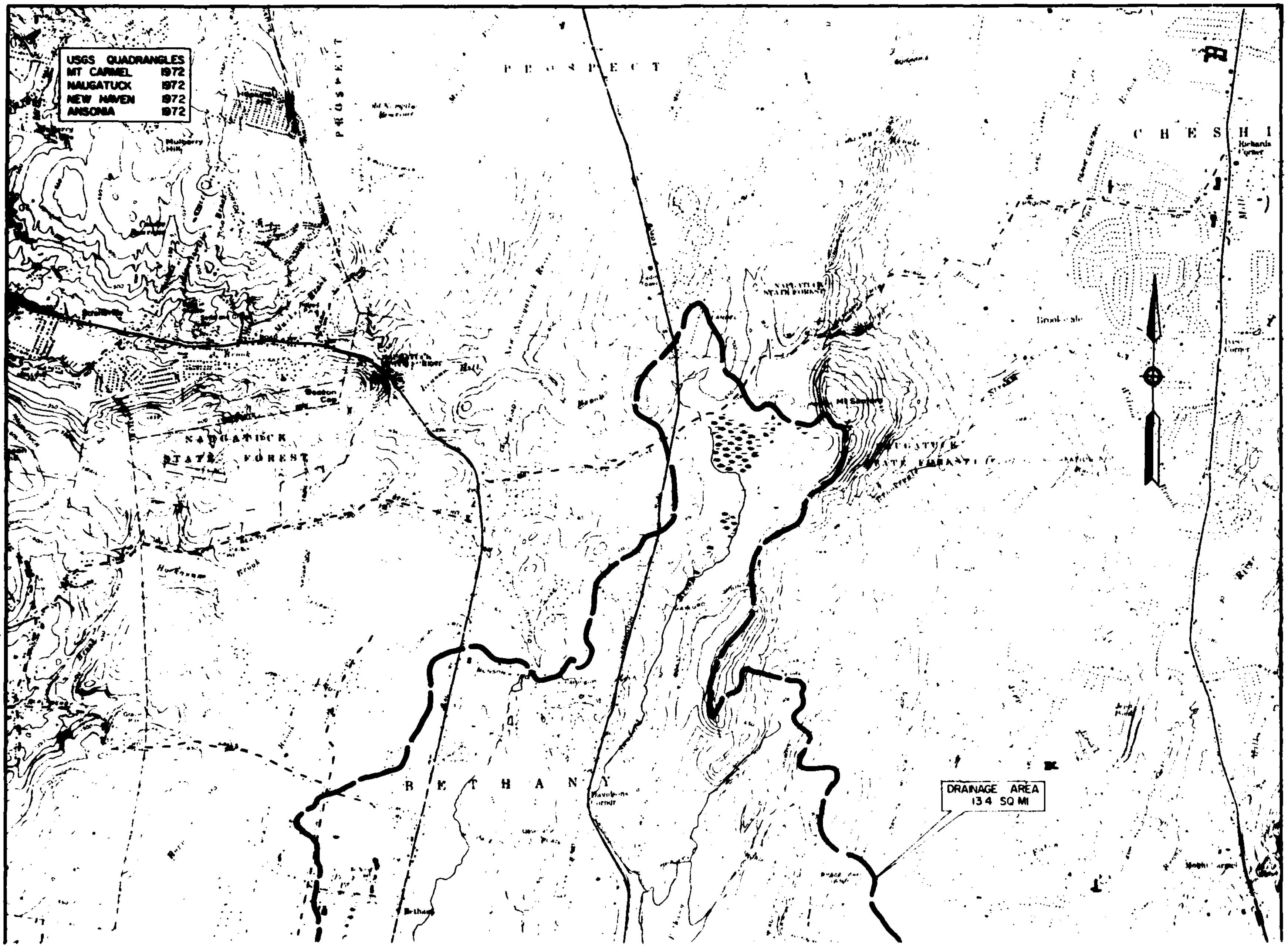
PHOTO 9 - Distress of concrete and seepage through central longitudinal crack on bottom of concrete spillway channel

PHOTO 10 - Undermining of spillway channel slab at left downstream end.

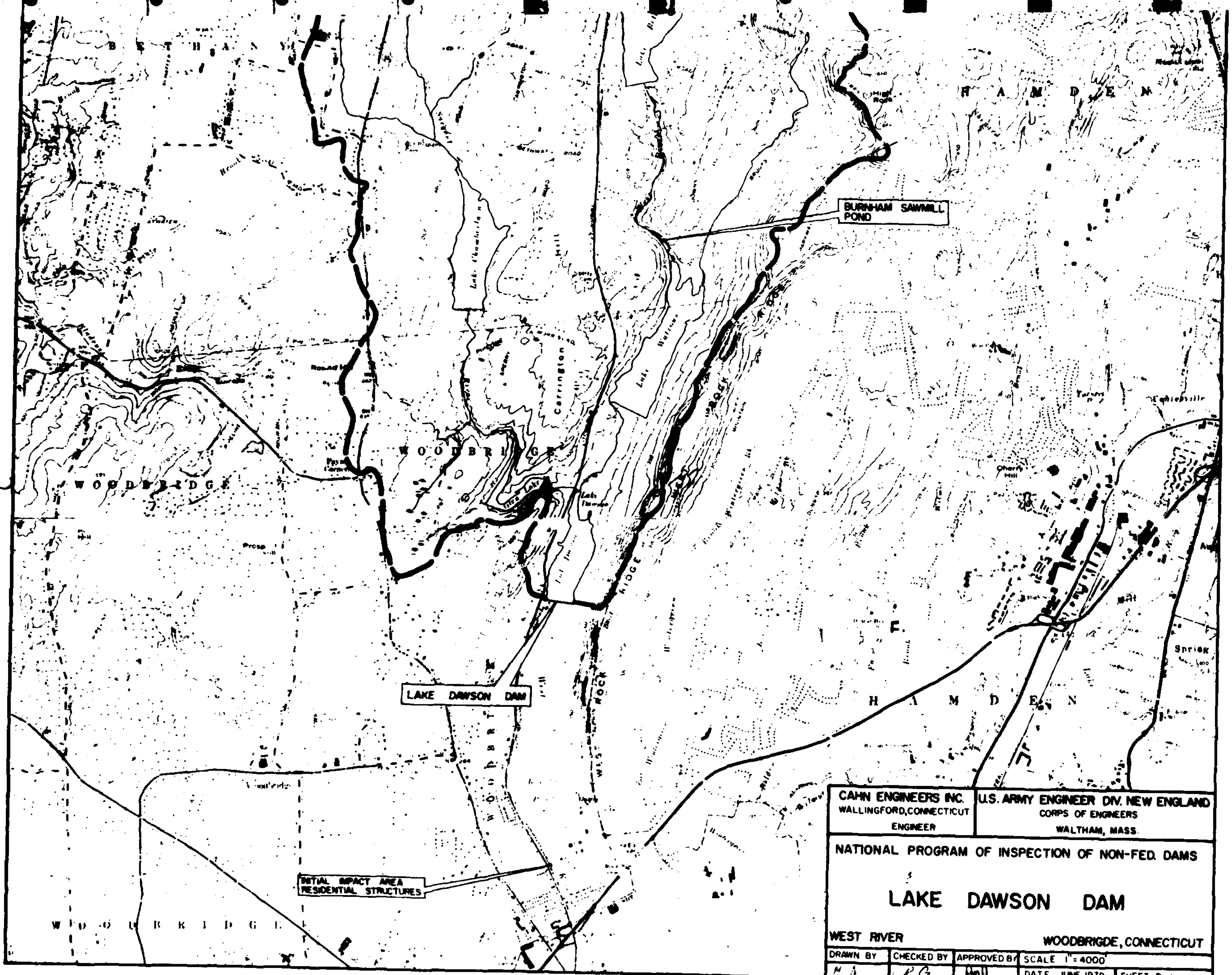
US ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.	NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS	Lake Dawson Dam West River Woodbridge, Connecticut
CAHN ENGINEERS INC. WALLINGFORD, CONN. ENGINEER		CE# 27 660 KA DATE May '79 PAGE C-5

APPENDIX D

HYDRAULICS/HYDROLOGIC COMPUTATIONS



USGS QUADRANGLES
MT CARMEL 1972
NAUGATUCK 1972
NEW HAVEN 1972
ANSONIA 1972
PROSPECT
CHESHIRE
NAUGATUCK STATE FOREST
BETHANY
DRAINAGE AREA
13.4 SQ MI



CAHN ENGINEERS INC. WALLINGFORD, CONNECTICUT ENGINEER	U.S. ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

LAKE DAWSON DAM

WEST RIVER — WOODBRIGDE, CONNECTICUT

DRAWN BY	CHECKED BY	APPROVED BY	SCALE 1" = 4000'	
M A	R G	Hall	DATE JUNE 1979	SHEET D-1

HYDROLOGIC/HYDRAULIC INSPECTION

LAKE DAWSON DAM, WOODBRIDGE, CT.

I) PERFORMANCE AT TEST FLOOD CONDITIONS:

1) MAXIMUM PROBABLE FLOOD:

a) WATERSHED CLASSIFIED AS "ROLLING" TO "FLAT"

b) WATERSHED AREA

i) TOTAL *D.A. ≅ 13.4 sq mi

ii) D.A. U/S FROM LAKE WATROUS: *D.A. ≅ 6.9 sq mi

iii) D.A. U/S FROM GLEN LAKE: *D.A. ≅ 5.7 sq mi

iv) DIRECT D.A. TO LAKE DAWSON (U/S FROM ABOVE DAMS): *D.A. ≅ 0.8 sq mi

*NOTE: DATA FROM NEW HAVEN WATER CO. (REPORT AND DATA BY J.W. COWE) ON THE DAMS ON WEST & SARGENT RIVERS, DATED JUNE, 1965, AND C.E. LAKE CHAMBERLAIN DAM, CT. 00306, AND LAKE WATROUS DAM CT 00318, PHASE I INSPECTION REPORTS DATED, AUGUST 1978.

c) FROM NED-ACE "PRELIMINARY GUIDANCE FOR ESTIMATING MAX PROBABLE DISCHARGES" - GUIDE CURVE FOR PMF - PEAK FLOW RATES.

i) PMF ≅ 1500 cfs/sq mi FOR TOTAL D.A.
ii) PMF ≅ 1800 cfs/sq mi FOR THE REGULATED D.A.'S.
iii) PMF ≅ 2300 cfs/sq mi FOR THE DIRECT AREA TO LAKE DAWSON (BY EXTRAPOLATION)

LAKE DAWSON DAM

1-CONT'D) MAXIMUM PROBABLE FLOOD

d) PEAK INFLOW

BECAUSE THE PMF PEAK REGULATION PRODUCED BY BOTH, LAKE WATROUS ($Q_{P_3} = 11400^{CFS}$) AND GLEN LAKE ($Q_{P_3} = 8200^{CFS}$) IS RELATIVELY SMALL ($(Q_{P_1})_{WAT.} = 12500^{CFS}$; $(Q_{P_1})_{GLEN} = 8600^{CFS}$), THE EFFECT OF THESE RESERVOIRS ON THE PMF PEAK INFLOW TO LAKE DAWSON IS NEGLECTED. THUS:

$$PMF = 13.4 \times 1500 = \underline{20100}^{CFS}$$

2) SPILLWAY DESIGN FLOOD (SDF)

a) CLASSIFICATION OF DAM ACCORDING TO NED-ACE RECOMMENDED GUIDELINES:

i) SIZE: STORAGE (MAX) $\cong 1540^{AC FT}$ $(1000 < S < 50000^{AC FT})$
HEIGHT $\cong 48'$ $(40 < H < 100^{FT})$

STORAGE: FROM DATA FURNISHED BY THE NEW HAVEN WATER CO.: "NEW HAVEN WATER CO. RESERVOIR CAPACITIES - WEST RIVER SYSTEM" BY A.B. HILL, REVISED TO MAR. 1923; SUMMARY DATA SHEET BY J.W.C. DATED JUNE, 1965 AND INVENTORY (STATISTICS ON DAMS DATED 8/12/74: RESERVOIR STORAGE TO *ELEV. 160.8' MSL (157.5' MHW) - TOP OF FLASHBOARDS - $S \cong 996.1 + 88.4 \cong 1085^{AC.FT}$ ($\pm 354^{MG}$); AREA AT NEW SPILLWAY CREST EL. 158.3' MSL (155' MHW) $A = 61.1^{AC}$; AREA AT TOP OF STOP PLANKS ELEV. 160.8' MSL, $A = 69.5^{AC}$; C.E. MEASURED AREA @ CONTOUR 170' MSL: $A_{170} = 92^{AC}$

*NOTE: ELEVATIONS GIVEN IN NEW HAVEN WATER CO. DATA ARE NEW HAVEN DATUM (MHW).
USCGS DATUM (MSL) ≡ NEW HAVEN DATUM (MHW) + 3.31'
(USE +3.3')

Cahn Engineers Inc. Consulting Engineers

Project NON-FEDERAL DAMS INSPECTION Sheet D-3 of 16
Computed By HW Checked By [illegible] Date 5/22/79
Field Book Ref. Other Refs. CE #27-660-KA Revisions

LAKE DAWSON DAM

2,a - Cont'd) CLASSIFICATION OF DAM ACCORDING TO NED-ACE GUIDELINES.

STORAGE (Cont'd): ∴ USE $A_{AVG} \approx 76^{AC}$ ∴ STORAGE TO TOP OF EMBANKMENT (EL. 166.8' MSL $\equiv$ 163.5' MHW): $S_{MAX} = 1085 + 6 \times 76 \approx \underline{1540}^{AC.FT}$

HEIGHT: FROM SAME DATA SOURCES AS FOR STORAGE; H = 48' IN DATA; ACTUALLY THIS HEIGHT IS FROM NATURAL STREAM BED ELEV. 119.3' MSL (116' MHW) TO ELEV. 167.3' MSL (164' MHW) WHICH IS THE TOP ELEV. OF THE SPILLWAY WALLS. THE EMBANKMENT IS (±) 0.5' LOWER (ELEV. 166.8' MSL ≡ 163.5' MHW) AND THEREFORE, THE ACTUAL HEIGHT (EXCEPT FOR A SHORT DISTANCE NEAR THE SPILLWAY WALLS) IS (±) 47.5' (ROUNDED TO 48'). HEIGHT TO DEEPEST FOUNDATION IS (±) 55' (SEE N.H.W.Co. INVENTORY).

ii) HAZARD POTENTIAL: LAKE DAWSON DAM IS LOCATED JUST U/S FROM THE WATER FILTRATION PLANT (NEW HAVEN WATER Co.). THE DAM IS ALSO LOCATED (±) 1 MI U/S FROM KONOLDS POND. NUMEROUS LOW HOUSES AND OTHER BUILDINGS WITH FIRST FLOOR ELEVATIONS WITHIN 5' ABOVE THE WATER LINE, ARE LOCATED AT THE SHORE OF KONOLDS POND AND ALONG WEST RIVER BOTH, U/S AND D/S FROM THE POND. FURTHER D/S, WEST RIVER ENTERS URBAN AREAS IN NEW HAVEN.

iii) CLASSIFICATION:

SIZE: INTERMEDIATE
HAZARD: HIGH

b) SDF = PMF = 20100^{CFS} $\frac{1}{2}$PMF $\approx 10000^{CFS}$

3) SURCHARGE AT PEAK INFLOW

a) PEAK INFLOW: $Q_{P_1} = 20100^{CFS}$ $Q'_{P_1} = \frac{1}{2}$ PMF $\approx 10000^{CFS}$

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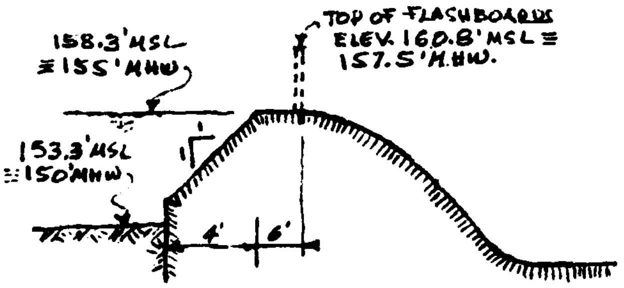
LAKE DAWSON DAM

3-(CONT'D) SURCHARGE AT PEAK INFLOW

b) SPILLWAY (OUTFLOW) RATING CURVE

i) SPILLWAY:

THE LAKE DAWSON DAM SPILLWAY IS CLASSIFIED AS A BROAD CRESTED COMPOUND WEIR WITH INCLINED U/S FACE ON 1"TO 1" SLOPE AND CURVED (WES) D/S FACE. THE U/S CURVED PORTION OF A STANDARD WES SHAPE HAS BEEN REPLACED BY A FLAT SECTION 2' IN BREADTH. THE CREST CAN ACCOMMODATE 2.5' HIGH FLASHBOARDS WHICH ARE DESIGNED TO FAIL AT A HEAD OF 2' (W.S. ELEV. 162.8' MSL ≡ 159.5' MHW). THE CREST IS 110' LONG AND SET AT ELEV. 158.3' MSL (155' MHW). THE DEPTH OF THE APPROACH CHANNEL TO THE CREST OF THE SPILLWAY IS P=5'.



THE FLASHBOARDS WERE REMOVED AFTER THEIR COLLAPSE DURING THE STORMS OF JAN. 1979. THE NEW HAVEN WATER CO. REPORTS NO INTENTION OF REINSTALLING THEM IN THE FUTURE. THEREFORE, THE PRESENT ANALYSIS WILL BE MADE PRIMARILY ON THE ASSUMPTION THAT NO FLASH BOARDS ARE IN PLACE, AS THIS IS THE PRESENT AND STATED FUTURE CONDITION OF THE STRUCTURE.

DATA FROM THE NEW HAVEN WATER CO. RECORD DRAWINGS (6/5/73) BY MALCOLM PIRNIE ENGRS. "LAKE DAWSON DAM - SPILLWAY MODIFICATIONS" DATED JUN. 1968.

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LAKE DAWSON DAM

3,b-CONT'D) OUTFLOW RATING CURVE

∴ ASSUME SPILLWAY DISCHARGE COEFFICIENT: $C \approx 3.6$

USING THE CREST ELEVATION AS DATUM (ELEV. 158.3' MSL = 155' MHW), THE SPILLWAY DISCHARGE IS APPROXIMATED BY:

$$Q_S \approx \underline{400\,H^{3/2}}$$

ii) EXTENSION OF RATING CURVE FOR SURCHARGE HEADS ABOVE TOP OF DAM.

THE DAM IS AN EARTH EMBANKMENT OF (±)18' TOP WIDTH AND 2^H TO 1^V U/S AND D/S FACE SLOPES. ALTHOUGH THE TOP OF THE SPILLWAY SIDE WALLS IS AT ELEV. 167.3' MSL (164' MHW), THE TOP OF THE EMBANKMENT IS GENERALLY AT ELEV. 166.8' MSL (163.5' MHW). THE LENGTH EXCLUDING THE SPILLWAY IS (±)* L = 800'.
THE TERRAIN TO THE RIGHT OF THE DAM RISES 10' IN A DISTANCE OF (±) 800'. TO THE LEFT, THE TERRAIN RISES 30' IN A DISTANCE OF (±) 150'. BOTH SIDES ARE FAIRLY CLEAR FROM TREES AND OTHER FLOW OBSTRUCTIONS. (DATA FROM AVAILABLE MAPS AND C.E. FIELD OBSERVATIONS).

ASSUME C = 3.0 FOR FLOW OVER THE TOP OF THE EMBANKMENT
C = 2.8 FOR FLOW OVER THE SLOPING TERRAIN

ASSUME ALSO EQUIVALENT LENGTHS FOR THE SLOPING TERRAIN AT THE SIDES OF THE DAM AND CORRESPONDING FLOW FORMULAS TO APPROXIMATE THE OVERFLOW AS FOLLOWS:

*NOTE: C.E. SCALED ON NEW HAVEN WATER CO. RECORD DWGS. (6/5/15) BY MALCOLM PIRNIE ENG. (SCALE 1":40'). CONN D.E.P. WATER RESOURCES ALSO GIVES L=800'; N.H.W.C. RECORD SHEET GIVES HOWEVER, L=±1000' INCL. THE SPILLWAY; J.W. CONE L=760'.

LAKE DAWSON DAM

3, b - CONT'D) OUTFLOW RATING CURVE:

LEFT: $L'_L = \frac{2}{3}\left(\frac{150}{30}\right)(H-8.5) = 3.3\,(H-8.5)$ ∴ $Q'_L \doteq 9.2\,(H-8.5)^{5/2}$

RIGHT: $L'_R = \frac{2}{3}\left(\frac{800}{10}\right)(H-8.5) = 53.3\,(H-8.5)$ $Q'_R \doteq 150\,(H-8.5)^{5/2}$

THEREFORE THE TOTAL OVERFLOW RATING CURVE MAY BE APPROXIMATED BY:

$$Q = Q_S + Q_D + Q'_{R+L} = 400H^{3/2} + 2400(H-8.5)^{3/2} + 160\,(H-8.5)^{5/2}$$

(FOR $H \leqq 18'$)

WHERE Q_S, Q_D AND Q'_{R+L} ARE THE FLOWS OVER THE SPILLWAY, DAM AND SLOPING TERRAIN AT THE SIDES OF THE DAM, RESPECTIVELY.

THE OUTFLOW RATING CURVE IS PLOTTED ON NEXT PAGE.

c) SPILLWAY CAPACITY TO TOP OF EMBANKMENT (EL. 166.8' MSL = 163.5 MHW)

$H = 8.5'$ ∴ $Q_S \doteq 9900^{cfs}$ ((±)49% OF Q_{P_1}; (±)99% OF Q'_{P_1})

NOTE: IF THE FLASHBOARDS ARE INSTALLED (C = 3.3) THE SPILLWAY CAPACITY TO TOP OF EMBANKMENT WILL NOT BE ACTUALLY REDUCED BECAUSE OF ITS COLLAPSE AT A DESIGN HEAD OF 2' (I.E. 4.5' ABOVE THE SPILLWAY CREST). THE DISCHARGE AT THE FAILURE OF THE FLASHBOARDS WILL BE (±)1000 CFS. OPERATION WITH FLASHBOARDS WILL REDUCE, HOWEVER, THE SURCHARGE STORAGE CAPACITY OF THE RESERVOIR.

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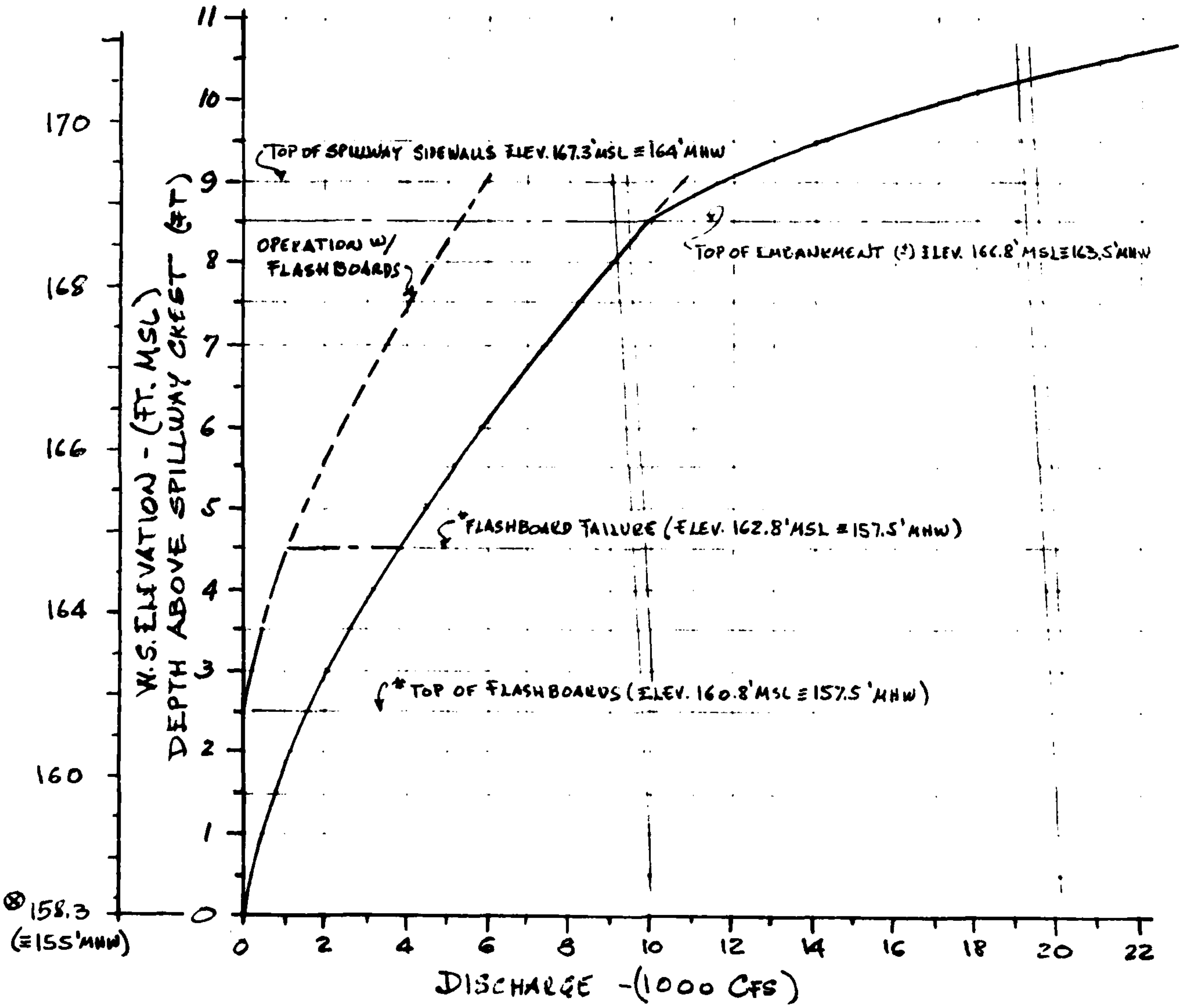
LAKE DAWSON DAM

(Cont'd) OUTFLOW RATING CURVE



*NOTE: OPERATION OF THE SPILLWAY WITH FLASHBOARDS IS DISCONTINUED (NEW HAVEN WATER CO.)

⊗SPILLWAY CREST ELEV. 158.3' MSL = 155' MHW

MSL (USCGS DATUM) = MHW (NEW HAVEN WATER CO. DATUM) + 3.3'

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LAKE DAWSON DAM

3-Cont'd) SURCHARGE AT PEAK INFLOW

d) SURCHARGE HEIGHT TO PASS (Q_{P_1}):

i) @ Q_{P_1} = PMF = 20100 cfs $H_1 \cong 10.4'$

ii) @ $Q_{P_1}' = \frac{1}{2}$PMF $\cong$ 10000 cfs $H_1' \cong 8.6'$

4) EFFECT OF SURCHARGE ON MAXIMUM PROBABLE DISCHARGES (OUTFLOW)

a) RESERVOIR (LAKE) AREA @ FLOW LINE: *A $\cong$ 61.1 ac (w/o FLASHBOARDS)

∴ ASSUME AVG LAKE AREA WITHIN EXPECTED SURCHARGE: *$A_{AVG} \cong 76$ ac

*SEE "STORAGE" pp. 2 & 3 OF THESE COMPUTATIONS.

b) ASSUME NORMAL POOL LEVEL (±) 0.5' ABOVE SPILLWAY CREST (EL. 158.8 msl)

c) WATERSHED AREA: D.A. = 13.4 sq mi (See p. 1)

d) DISCHARGE (Q_{P_2}) AT VARIOUS HYPOTHETICAL SURCHARGE ELEVATIONS:

$H = 9'$ $V = 76(9-0.5) \cong 646$ AC-FT ∴ $S = \frac{646}{13.4 \times 53.3} = 0.90"$

$H = 4'$ $V = 266$ AC-FT ∴ $S = 0.37"$

∴ FROM APPROXIMATE STORAGE ROUTING NED-ACE GUIDELINES: (19" MAX. PROBABLE R.O. IN NEW ENGLAND):

$Q_{P_2} = Q_{P_1}\left(1 - \frac{S}{19}\right)$ AND FOR ½ PMF $Q_{P_2}' = Q_{P_1}'\left(1 - \frac{S}{9.5}\right)$

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LAKE DAWSON DAM

4,d-(Cont'd) EFFECT OF SURCHARGE STORAGE ON PEAK OUTFLOW:

∴ FOR THE PREVIOUS HYPOTHETICAL SURCHARGES:

$H = 9'$ $Q_{p_2} = 17100^{cfs}$ $Q'_{p_2} = 7100^{cfs}$

$H = 4'$ $Q_{p_2} = 19700^{cfs}$ $Q'_{p_2} = 7500^{cfs}$

ALSO, FOR $H = 0.5'$; $Q_{p_2} = 20100^{cfs}$ AND $Q'_{p_2} = 10000^{cfs}$

e) PEAK OUTFLOW (Q_{p_3})

USING NED-ACE GUIDELINES "SURCHARGE STORAGE ROUTING" ALTERNATE METHOD (SEE p. 7 OF THESE CALCS):

$Q_{p_3} \cong 19000^{cfs}$ $H_3 \cong 10.2'$ FOR Q_{p_1} = PMF

$Q'_{p_3} \cong 9200^{cfs}$ $H'_3 \cong 8.1'$ FOR Q'_{p_1} = ½ PMF

IT SHOULD BE NOTED THAT LAKE DAWSON'S SURCHARGE STORAGE HAS RELATIVELY LITTLE EFFECT ON THE REDUCTION OF THE TEST FLOOD PEAK INFLOW. IF THE FLASHBOARDS ARE REINSTALLED, ASSUMING THAT FAILURE OCCURS AS DESIGNED, AT A 2' HEAD, THE SPILLWAY FULL CAPACITY WILL BE RESTORED (SEE p. 7) WITH THE DAM STILL HAVING A REASONABLE FREEBOARD. IF THE TEST FLOOD FOR OPERATION WITH FLASHBOARDS IS ASSUMED REACHING THE RESERVOIR AT AN ASSUMED NORMAL POOL, SAY, 0.5' ABOVE THE TOP OF THE FLASHBOARDS (IE. 3' ABOVE THE SPILLWAY CREST), THE CONDITIONS WILL BE SIMILAR TO THOSE ANALIZED, EXCEPT FOR A RELATIVELY SMALL REDUCTION OF SURCHARGE STORAGE (±170 AC FT). THEREFORE, THE EXPECTED INCREASE IN PEAK OUTFLOW (Q_{p_3}) AND CORRESPONDING SURCHARGE (H_3) FOR CONDITIONS W/FLASHB. IS NEGLIGEABLE.

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LAKE DAWSON DAM

4. (CONT'D) EFFECT OF SURCHARGE STORAGE ON PEAK OUTFLOW

f) SPILLWAY CAPACITY RATIO TO OUTFLOW

SPILLWAY CAPACITY TO TOP OF EMBANKMENT: $Q_S \doteq 9900^{cfs}$

∴ SPILLWAY CAPACITY IS (±) 52% THE OUTFLOW @ PMF AND (±)110% THE OUTFLOW @ 1/2 PMF.

5) SUMMARY:

a) PEAK INFLOW: $Q_{P_1} = PMF = 20100^{cfs}$ $Q'_{P_1} = \frac{1}{2} PMF \doteq 10000^{cfs}$

b) PEAK OUTFLOW: $Q_{P_3} \doteq 19000^{cfs}$ $Q'_{P_3} \doteq 9200^{cfs}$

c) SPILLWAY MAX. CAPACITY $Q_S \doteq 9900^{cfs}$ OR, (±)52% OF Q_{P_3} AND (±)110% OF Q'_{P_3}

THEREFORE, AT SDF = PMF, THE DAM IS OVERTOPPED (±) 1.7' (W.S. ELEV. 168.5' MSL ≡ 165.2' MHW) OR, TO A SURCHARGE OF (±)10.2' ABOVE THE SPILLWAY CREST.
AT A TEST FLOOD $Q'_{P_1} = \frac{1}{2} PMF$, THE SPILLWAY MAY PASS THE OUTFLOW WITH A FREEBOARD OF (±) 0.4' (W.S. EL. 166.4' MSL ≡ 163.1' MHW) OR, TO A SURCHARGE OF (±)8.1' ABOVE THE SPILLWAY CREST.
SIMILAR OVERFLOWS/SURCHARGES ARE EXPECTED FOR THE OPERATION OF THE SPILLWAY WITH FLASHBOARDS.

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LAKE DAWSON DAM

II) DOWNSTREAM FAILURE HAZARD

1) PEAK FLOOD AND STAGE IMMEDIATELY D/S FROM DAM:

a) BREACH WIDTH:

i) MID HEIGHT (±) ELEV. 142.8' MSL (139.5' MHW) $(166.8 - \frac{*48}{2} = 142.8' MSL)$

*SEE "HEIGHT" p.3 OF THESE COMPS.

∴ ii) APPROX. MID-HEIGHT LENGTH: $\ell \approx 600'$ ((±) C.E. MEASURE ON M. PIERCE'S "LAKE DAWSON DAM" SPWY. MODIF. MAT JUNE '68

∴ iii) BREACH WIDTH (SEE NED-ACE D/S DAM FAILURE GUIDELINES):

$$W = 0.4 \times 600 = 240' \therefore \text{ASSUME } W_b \approx \underline{\underline{240'}}$$

b) PEAK FAILURE OUTFLOW (Q_{p_1})

ASSUME SURCHARGE TO TOP OF DAM (EMBANKMENT); THEREFORE,

i) HEIGHT AT TIME OF FAILURE: $Y_0 = 47.5'$ (SEE p.3 OF THESE COMPS.)

ii) SPILLWAY DISCHARGE: $Q_S \approx 9900^{CFS}$

iii) BREACH OUTFLOW (Q_b):

$$Q_b = \frac{8}{27} W_b \sqrt{g}\, Y_0^{3/2} \approx 132,100^{CFS}$$

iv) PEAK FAILURE OUTFLOW (Q_{p_1}): $Q_{p_1} = Q_S + Q_b \approx \underline{\underline{142000^{CFS}}}$

c) FLOOD WAVE HEIGHT IMMEDIATELY D/S FROM DAM:

$$Y \approx 0.44\, Y_0 \approx \underline{\underline{21'}}$$

LAKE DAWSON DAM

2) ESTIMATE OF D/S DAM FAILURE CONDITIONS AT IMPACT AREA:

(SEE NED-ACE GUIDELINES FOR ESTIMATING D/S DAM FAILURE HYDROGRAPHS)

LAKE DAWSON IS LOCATED (±) 1 MI U/S FROM KONOLDS POND. THE CHANNEL BETWEEN THE TWO RESERVOIRS IS A VERY WIDE, LOW, SWAMPY AREA WHICH EXCEPT PERHAPS FOR THE FIRST 1000', SLOPES VERY GENTLY INTO KONOLDS POND.

THEREFORE, BECAUSE OF THE LARGE STORAGE CAPACITY OF THE VALLEY CONNECTING THE RESERVOIRS (ACTUALLY SURCHARGE STORAGE OF KONOLDS POND) A SIMPLIFICATION WILL BE MADE BY ONLY ROUTING THE LAKE DAWSON'S DAM FAILURE FLOOD FLOW THROUGH KONOLDS POND. IT IS OBSERVED THAT ALTHOUGH SOME ATTENUATION OF THE FLOOD FLOW IS EXPECTED, DUE TO STORAGE IN THE STEEPER (±) 1500' U/S REACH, THE NET EFFECT ON THE STAGE D/S FROM KONOLDS POND WILL BE NEGLIGEABLE BECAUSE OF THE VERY LARGE SURCHARGE STORAGE CAPACITY OF THIS POND.

a) W.L. RAISE AND OUTFLOW AT KONOLDS POND BECAUSE OF FAILURE OF LAKE DAWSON DAM:

i) VOLUME OF STORAGE AT TIME OF FAILURE (DAWSON): $S = 1540^{AC\text{-}FT}$
(SEE p. 2 OF THESE COMPUTATIONS - RESERVOIR FULL TO TOP OF DAM)

ii) ASSUMED PEAK INFLOW TO KONOLDS POND: $Q_{P_1} = \underline{142000}$ CFS

iii) KONOLDS POND DATA (FROM C.E. FIELD OBSERVATIONS ON 4/23/79)

KONOLDS POND DAM IS AN EARTH EMBANKMENT (±) 300' LONG, EXCLUDING A 60' LONG CONCRETE TRAPEZOIDAL BROADCRESTED SILL.

LAKE DAWSON DAM

2,a-Cont'd) D/S DAM FAILURE CONDITIONS - (KONOLDS POND DATA)

SPILLWAY WITH (±) 1' DEPTH TO THE TOP OF THE DAM. THE EMBANKMENT SHOWS SIGNS OF SCOUR PROBABLY CAUSED BY OVERTOPPING.

FOR THE PURPOSE OF THIS CALCULATION THE SPILLWAY CAPACITY WILL BE IGNORED AND THE FULL LENGTH OF THE DAM PLUS PORTIONS OF THE ADJACENT TERRAIN WILL BE CONSIDERED AS THE OVERFLOW SECTION OF KONOLDS POND.

ASSUMING WL ELEV. 83' MSL (USGS, NEW HAVEN, CT. QUADRANGLE SHEET) AS THE OVERFLOW ELEVATION, THE TERRAIN AT THE RIGHT SIDE OF THE DAM SLOPES TO ELEV. 90' MSL IN (±) 150' AND THEN, GRADUALLY RAISES TO ELEV. 100' MSL IN (±) 1250'. TO THE LEFT, THE TERRAIN RISES SHARPLY TO ELEV. 90' MSL AND THEN, GRADUALLY, TO ELEV. 100' MSL IN (±) 900'.

THE POND STORAGE FOR SURCHARGES ABOVE ELEV. 83' MSL, ESTIMATED FROM AREA MEASURES AT VARIOUS CONTOUR ELEVATIONS ON THE U.S.G.S., NEW HAVEN, CT. QUADRANGLE (1:24000) IS PLOTTED ON NEXT PAGE.

ASSUME A DISCHARGE COEFFICIENT C=3.0 FOR THE ENTIRE OVERFLOW SECTION AND EQUIVALENT LENGTHS (AND CORRESPONDING DISCHARGE) FOR THE SLOPING PORTIONS OF TERRAIN, AS FOLLOWS:

$$L_R' = \frac{2}{3} \times 150 = 100' \text{ (CONSTANT)} \; \therefore \; Q_R' = 300 H^{3/2}$$

FOR THE 7' RISE TO THE RIGHT OF THE DAM

$$L_R'' = \frac{2}{3}\left(\frac{1250}{10}\right)(H-7) \; \therefore \; Q_R'' = 250(H-7)^{5/2}$$ FOR THE CONTINUING SLOPING TERRAIN TO THE RIGHT.

LAKE DAWSON DAM

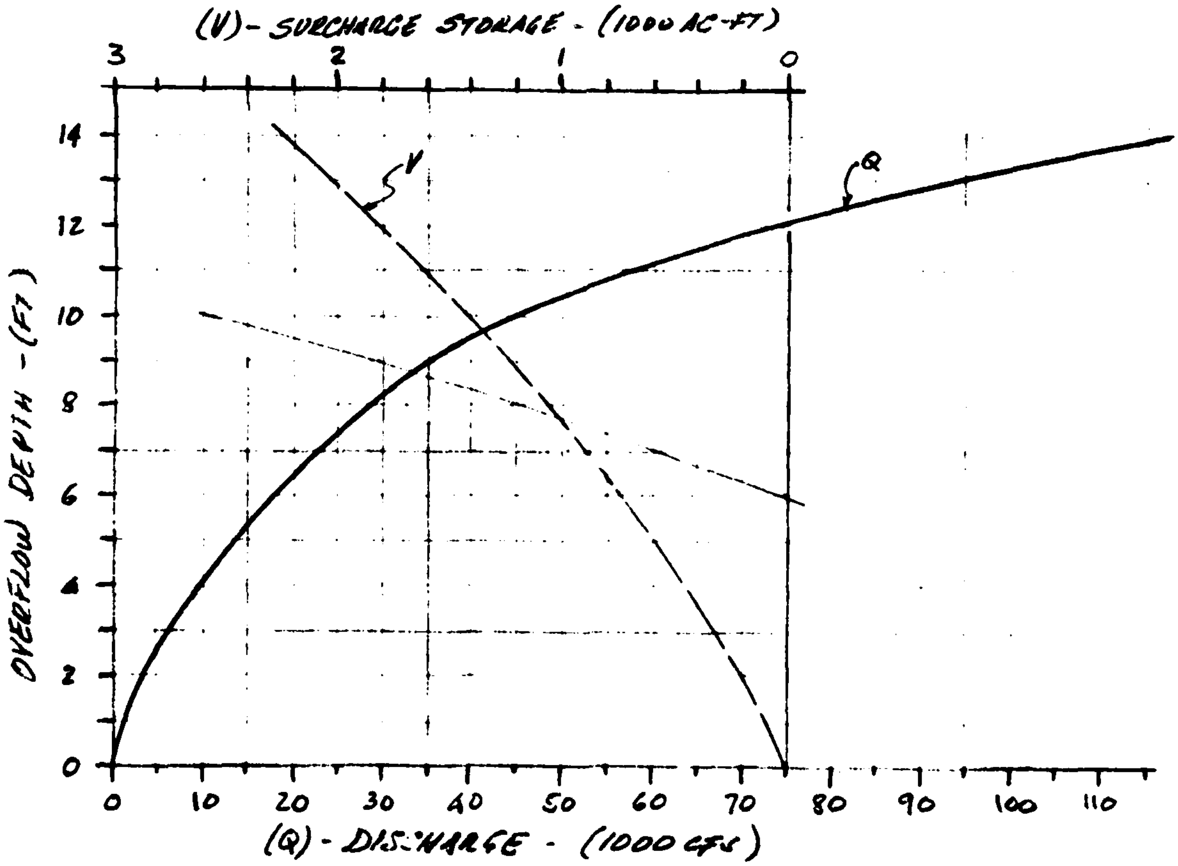
2,a - Cont'd.) b/ DAM FAILURE CONDITIONS - (KONOLS POND DATA)

AND, $L_L' = \frac{2}{3}\left(\frac{900}{10}\right)(H-7)$ $\therefore$ $Q_L' = 180\,(H-7)^{5/2}$ FOR THE SLOPING TERRAIN TO THE LEFT ABOVE H=7'.

$\therefore$ THE KONOLS POND OUTFLOW CAN BE APPROXIMATED BY:

$$Q_K = 1200\,H^{3/2} + 430\,(H-7)^{5/2}$$ (THE 2nd TERM ONLY FOR H > 7')

(ii) KONOLS POND SURCHARGE STORAGE AND OVERFLOW RATING CURVES:



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LAKE DAWSON DAM

2-CONT'D) D/S DAM FAILURE CONDITIONS

b) KONOLDS POND W.L. AND OUTFLOW

THE OVERFLOW WILL BE ESTIMATED FOLLOWING A SIMILAR PROCEDURE TO THAT USED IN ROUTING THE PMF TEST FLOOD THROUGH LAKE DAWSON (SEE pp. 7-9 OF THESE COMPS)

i) DISCHARGE (Q_{P_2}) AT VARIOUS HYPOTHETICAL SURCHARGES (KONOLDS):

$H = 9'$ $\quad V \approx 1220^{AC \cdot FT}$ $\quad \therefore Q_{P_2} = 142000\left(1 - \frac{1220}{1540}\right) \approx 29500^{CFS}$

$H = 8'$ $\quad V \approx 1060^{ACFT}$ $\quad \therefore Q_{P_2} \approx 44300^{CFS}$

$H = 7'$ $\quad V \approx 880^{ACFT}$ $\quad \therefore Q_{P_2} \approx 60700^{CFS}$

$H = 6'$ $\quad V \approx 730^{ACFT}$ $\quad \therefore Q_{P_2} \approx 74700^{CFS}$

ii) KONOLDS POND OUTFLOW (Q_{P_3}) AND SURCHARGE (H_3):

$\therefore Q_{P_3} \approx 32000^{CFS}$ $\quad H_3 \approx 8.8'$ SAY, 9' (SEE p. 14 OF THESE COMPS)

THEREFORE, UPON FAILURE OF LAKE DAWSON DAM, KONOLDS POND DAM WOULD BE OVERTOPPED BY APPROX. 9' ((±)EL. 93' MSL) WITH A RESULTING OUTFLOW OF (±) $Q_{P_3} \approx 32000^{CFS}$ TO THE DOWNSTREAM IMPACT AREA ALONG THE WEST RIVER.

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LAKE DAWSON DAM

3) SUMMARY

a) PEAK FAILURE OUTFLOW: $Q_{p1} = 142000$ CFS $y_0 \doteq 21'$

b) KONOLDS POND DAM OVERFLOW: $Q_{p3} = 32000$ CFS

SURCHARGE (APPROX. OVERTOPPING) OF KONOLDS POND DAM: $H_3 \doteq 9'$

THEREFORE, FAILURE OF LAKE DAWSON DAM MAY AFFECT IMMEDIATELY D/S, THE WATER FILTRATION PLANT (F.F. ELEV. (±)135' MSL) AND SUBSEQUENTLY, LOW HOUSING AND BUILDINGS ALONG THE SHORE OF KONOLDS POND. THE KONOLDS POND DAM MAY BE OVERTOPPED BY (±) 9' AND WOULD DISCHARGE INTO THE D/S CHANNEL (HEAVILY URBANIZED WITH LOW HOUSING) WITH A PEAK FLOOD OF (±) 32000 CFS.

PRELIMINARY GUIDANCE

FOR ESTIMATING

MAXIMUM PROBABLE DISCHARGES

IN

PHASE I DAM SAFETY

INVESTIGATIONS

New England Division
Corps of Engineers

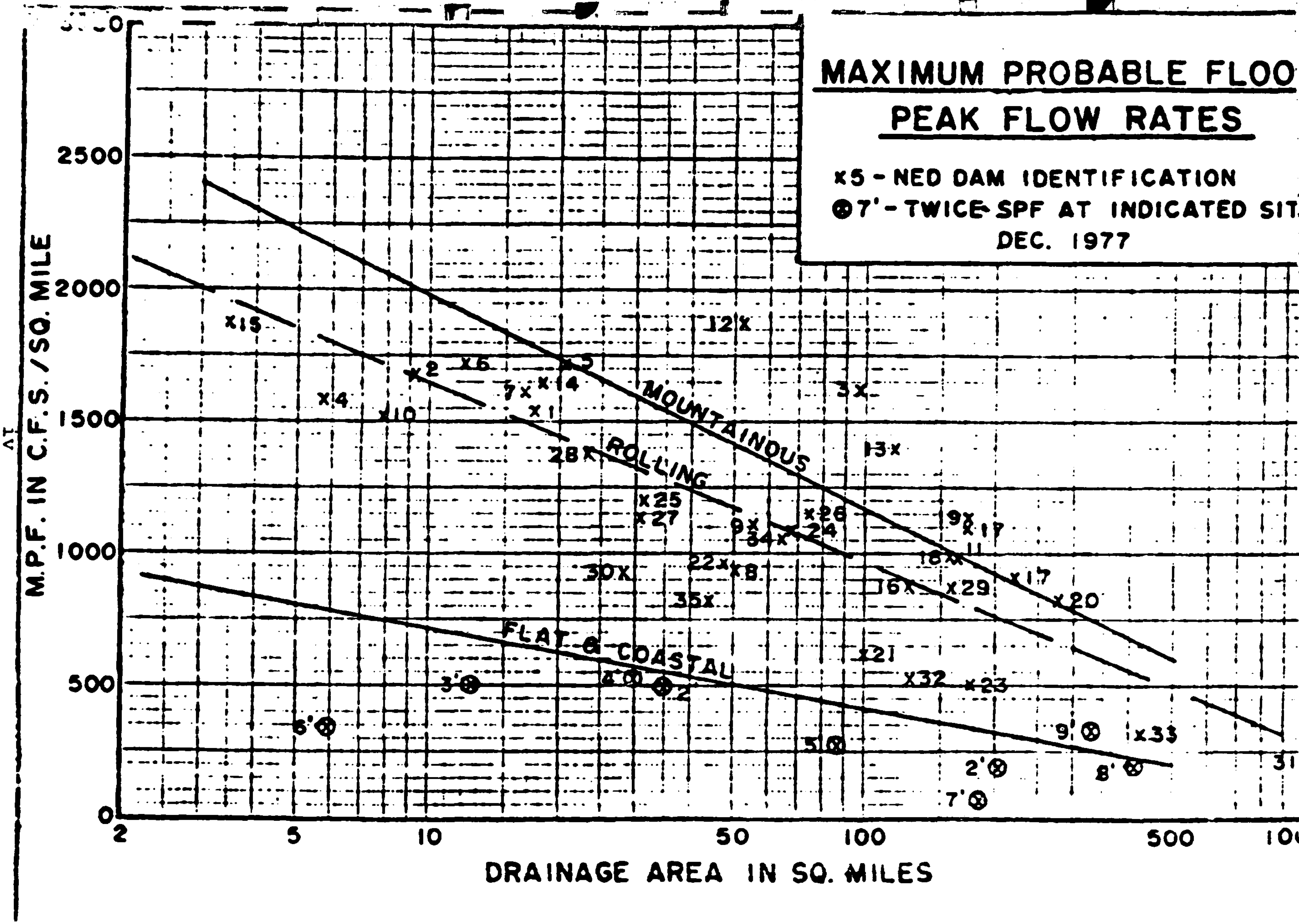
March 1978

MAXIMUM PROBABLE FLOOD INFLOWS
NED RESERVOIRS

	Project	Q (cfs)	D.A. (sq. mi.)	MPF cfs/sq. mi.
1.	Hall Meadow Brook	26,600	17.2	1,546
2.	East Branch	15,500	9.25	1,675
3.	Thomaston	158,000	97.2	1,625
4.	Northfield Brook	9,000	5.7	1,580
5.	Black Rock	35,000	20.4	1,715
6.	Hancock Brook	20,700	12.0	1,725
7.	Hop Brook	26,400	16.4	1,610
8.	Tully	47,000	50.0	940
9.	Barre Falls	61,000	55.0	1,109
10.	Conant Brook	11,900	7.8	1,525
11.	Knightville	160,000	162.0	987
12.	Littleville	98,000	52.3	1,870
13.	Colebrook River	165,000	118.0	1,400
14.	Mad River	30,000	18.2	1,650
15.	Sucker Brook	6,500	3.43	1,895
16.	Union Village	110,000	126.0	873
17.	North Hartland	199,000	220.0	904
18.	North Springfield	157,000	158.0	994
19.	Ball Mountain	190,000	172.0	1,105
20.	Townshend	228,000	106.0(278 total)	820
21.	Surry Mountain	63,000	100.0	630
22.	Otter Brook	45,000	47.0	957
23.	Birch Hill	88,500	175.0	505
24.	East Brimfield	73,900	67.5	1,095
25.	Westville	38,400	99.5(32 net)	1,200
26.	West Thompson	85,000	173.5(74 net)	1,150
27.	Hodges Village	35,600	31.1	1,145
28.	Buffumville	36,500	26.5	1,377
29.	Mansfield Hollow	125,000	159.0	786
30.	West Hill	26,000	28.0	928
31.	Franklin Falls	210,000	1000.0	210
32.	Blackwater	66,500	128.0	520
33.	Hopkinton	135,000	426.0	316
34.	Everett	68,000	64.0	1,062
35.	MacDowell	36,300	44.0	825

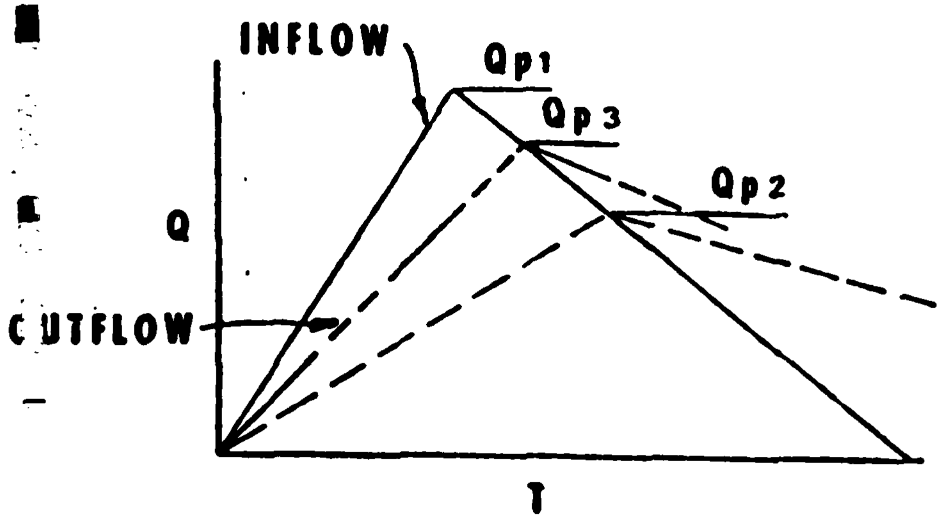
MAXIMUM PROBABLE FLOWS
BASED ON TWICE THE
STANDARD PROJECT FLOOD
(Flat and Coastal Areas)

	River	SPF (cfs)	D.A. (sq. mi.)	MPF (cfs/sq. mi.)
1.	Pawtuxet River	19,000	200	190
2.	Mill River (R.I.)	8,500	34	500
3.	Peters River (R.I.)	3,200	13	490
4.	Kettle Brook	8,000	30	530
5.	Sudbury River.	11,700	86	270
6.	Indian Brook (Hopk.)	1,000	5.9	340
7.	Charles River.	6,000	184	65
8.	Blackstone River.	43,000	416	200
9.	Quinebaug River	55,000	331	330



MAXIMUM PROBABLE FLOO
PEAK FLOW RATES
x5 - NED DAM IDENTIFICATION
⊗7' - TWICE SPF AT INDICATED SIT
DEC. 1977
MOUNTAINOUS
ROLLING
FLAT & COASTAL
M.P.F. IN C.F.S./SQ. MILE
DRAINAGE AREA IN SQ. MILES
0
500
1000
1500
2000
2500
2
5
10
50
100
500

ESTIMATING EFFECT OF SURCHARGE STORAGE ON MAXIMUM PROBABLE DISCHARGES



STEP 1: Determine Peak Inflow (Q_{p1}) from Guide Curves.

STEP 2: a. Determine Surcharge Height To Pass "Q_{p1}".

b. Determine Volume of Surcharge ($STOR_1$) In Inches of Runoff.

c. Maximum Probable Flood Runoff In New England equals Approx. 19", Therefore

$$Q_{p2} = Q_{p1} \times \left(1 - \frac{STOR_1}{19}\right)$$

STEP 3: a. Determine Surcharge Height and "$STOR_2$" To Pass "Q_{p2}"

b. Average "$STOR_1$" and "$STOR_2$" and Determine Average Surcharge and Resulting Peak Outflow "Q_{p3}".

SURCHARGE STORAGE ROUTING SUPPLEMENT

STEP 3: a. Determine Surcharge Height and "$STOR_2$" To Pass "Qp_2"

b. Avg "$STOR_1$" and "$STOR_2$" and Compute "Qp_3".

c. If Surcharge Height for Qp_3 and "$STOR_{AVG}$" agree O.K. If Not:

STEP 4: a. Determine Surcharge Height and "$STOR_3$" To Pass "Qp_3"

b. Avg. "Old $STOR_{AVG}$" and "$STOR_3$" and Compute "Qp_4"

c. Surcharge Height for Qp_4 and "New $STOR_{Avg}$" should Agree closely

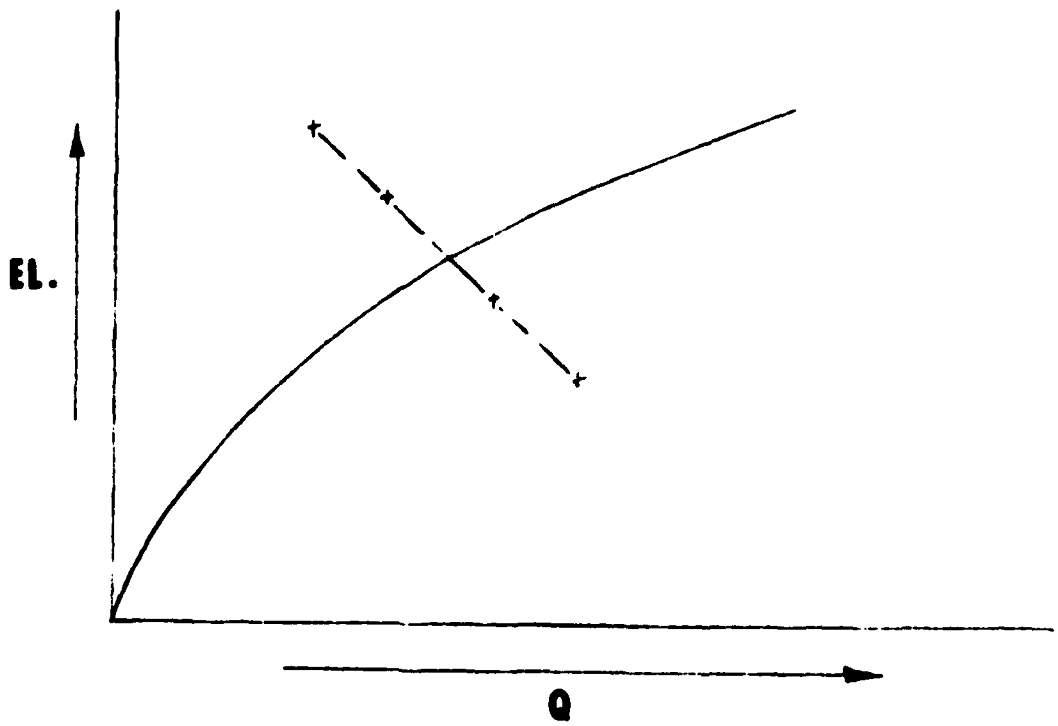
SURCHARGE STORAGE ROUTING ALTERNATE

$$Q_{p2} = Q_{p1} \times \left(1 - \frac{STOR}{19}\right)$$

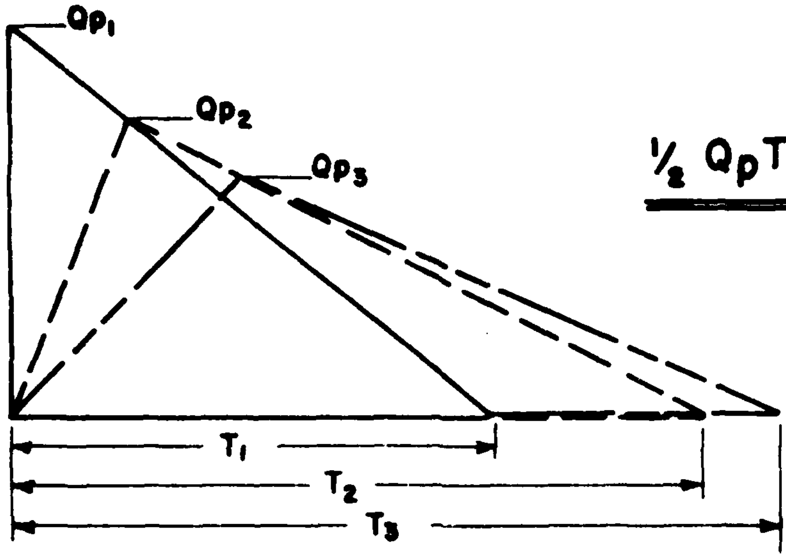
$$Q_{p2} = Q_{p1} - Q_{p1}\left(\frac{STOR}{19}\right)$$

FOR KNOWN Q_{p1} AND 19'' R.O.

Q_{p2}	STOR	EL.



"RULE OF THUMB" GUIDANCE FOR ESTIMATING DOWNSTREAM DAM FAILURE HYDROGRAPHS



$$\frac{1}{2} Q_p T = 12\, S$$

STEP 1: DETERMINE OR ESTIMATE RESERVOIR STORAGE (S) IN AC-FT AT TIME OF FAILURE.

STEP 2: DETERMINE PEAK FAILURE OUTFLOW (Q_{p1}).

$$Qp_1 = \frac{8}{27} W_b \sqrt{g}\, Y_0^{3/2}$$

W_b = BREACH WIDTH - SUGGEST VALUE NOT GREATER THAN 40% OF DAM LENGTH ACROSS RIVER AT MID HEIGHT.

Y_0 = TOTAL HEIGHT FROM RIVER BED TO POOL LEVEL AT FAILURE.

STEP 3: USING USGS TOPO OR OTHER DATA, DEVELOP REPRESENTATIVE STAGE-DISCHARGE RATING FOR SELECTED DOWNSTREAM RIVER REACH.

STEP 4: ESTIMATE REACH OUTFLOW (Q_{p2}) USING FOLLOWING ITERATION.

A. APPLY Q_{p1} TO STAGE RATING, DETERMINE STAGE AND ACCOPMANYING VOLUME (V_1) IN REACH IN AC-FT. (NOTE: IF V_1 EXCEEDS 1/2 OF S, SELECT SHORTER REACH.)

B. DETERMINE TRIAL Q_{p2}.

$$Qp_2(\text{TRIAL}) = Qp_1 \left(1 - \frac{V_1}{S}\right)$$

C. COMPUTE V_2 USING Q_{p2} (TRIAL).

D. AVERAGE V_1 AND V_2 AND COMPUTE Q_{p2}.

$$Qp_2 = Qp_1 \left(1 - \frac{V_{avg}}{S}\right)$$

STEP 5: FOR SUCCEEDING REACHES REPEAT STEPS 3 AND 4.

APRIL 1978

APPENDIX E

INFORMATION AS CONTAINED IN
THE NATIONAL INVENTORY OF DAMS

INVENTORY OF DAMS IN THE UNITED STATES

STATE	IDENTITY NUMBER	DIVISION	STATE	COUNTY	CONGR. DIST.	STATE	COUNTY	CONGR. DIST.	NAME	LATITUDE (NORTH)	LONGITUDE (WEST)	REPORT DATE DAY MO YR
CT	319	NED	CT	009	03				LAKE DAWSON DAM	4122.0	7258.7	31AUG79

POPULAR NAME	NAME OF IMPOUNDMENT
	LAKE DAWSON

REGION	BASIN	RIVER OR STREAM	NEAREST DOWNSTREAM CITY-TOWN-VILLAGE	DIST FROM DAM (MI.)	POPULATION
01	07	WEST RIVER	WOODBRIDGE	2	4000

TYPE OF DAM	YEAR COMPLETED	PURPOSES	STRUCTURAL HEIGHT (FT.)	HYDRAULIC HEIGHT (FT.)	IMPOUNDING CAPACITIES MAXIMUM (ACRE-FT.)	IMPOUNDING CAPACITIES NORMAL (ACRE-FT.)	DIST	OWN	FED R	PRV/FED	SCS A	VER/DATE
REPG	1890	S	55	48	1540	1080	NED	N	N	N	N	

REMARKS
21-WITH CONCRETE COREWALL

D/S HAS	SPILLWAY CREST LENGTH	SPILLWAY TYPE	SPILLWAY WIDTH (FT.)	MAXIMUM DISCHARGE (FT.)	VOLUME OF DAM (CY)	POWER CAPACITY INSTALLED (MW)	POWER CAPACITY PROPOSED (MW)	NAVIGATION LOCKS NO	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)
1	960	U	110	9900												

OWNER	ENGINEERING BY	CONSTRUCTION BY
NEW HAVEN WATER COMPANY	LUCIAN A TAYLOR	NEW HAVEN WATER COMPANY

REGULATORY AGENCY DESIGN	CONSTRUCTION	OPERATION	MAINTENANCE
CT WATER RESOURCES	CT WATER RESOURCES	CT WATER RESOURCES	CT WATER RESOURCES

INSPECTION BY	INSPECTION DATE DAY MO YR	AUTHORITY FOR INSPECTION
CAHN ENGINEERS INC	01MAY79	PL 92-367

REMARKS

END

FILMED

DTIC